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GROUP EFFECT ON UPLIFT CAPACITY OF DRILLED PIERS IN TILL

P.S. Toth

Narrow based lattice type transmission towers require constructing closely spaced footings which could experience reduction in uplift capacity due to group effect. This report summarizes the results of uplift tests on single footings and footing pairs including the load test of a four footing arrangement with two footings in compression and two footings in uplift. Analysis of the test data showed a reduction in the uplift coefficient Ku value by 36 per cent due to the overlapping zones of influence for the uplift footings. No reduction in the bearing capacity was observed.

INTRODUCTION

Narrow based towers are being considered for use in areas of restricted right-of-ways and to decrease the amount of land taken away from production in agricultural areas.

Single steel poles have been used in areas where the improved appearance is needed in addition to the efficient use of right-of-way. The cost of these steel poles are prohibitive for their use in all areas. Therefore a new design of narrow based lattice towers are being considered. The base of these towers are 3.35 m square. The foundations for this type of narrow based tower are required to resist large loads caused by wind and ice on the conductors and structure. Two possible schemes of the foundations for the narrow based tower are (i) four single footings, one for each tower leg and (ii) one single foundation for the lattice tower.

This report considers foundations under the first category; four cylindrical augered footings, 1.83 m in diameter are proposed. The wind loading on a tangent tower puts two footings in compression and two

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footings in uplift, which could reverse with the direction of wind. In the case of an angle tower or turning tower, the footings will be under sustained loading of compression or uplift depending on the orientation of tower legs to the resultant forces.

The possibility of a group effect for footings in uplift has previously been suggested/1/. Field testing of footing pairs in compact to dense sand/2/ have produced reduction factors due to group effect that are in agreement with the earlier laboratory and field tests in similar soils/2/. The group effect on uplift capacity for footings in clay have been studied with laboratory scale model footings/3/, but the results have not been verified in the field. There does not appear to be any information in the literature concerning the group effect for footings in till.

The proposed 500 kV transmission line from Bruce NGS to London TS traverses through areas mainly covered with dense till.

At the request of Mr. A.R. Collins of Stations and Transmission Design, the Civil Research Department carried out field load tests with assistance from the Mechanical Research Department, on closely spaced drilled piers in sandy silt till at the Milton Yard to assess the group effect on uplift capacity as related to narrow based tower foundation design.

Testing Program

The testing program was designed to address the following concerns:

1) Constructability of four large diameter, closely spaced augered pier foundations in dense sandy silt till.

2) Ultimate load and load-deflection behaviour of
   i) single footings
   ii) footing pairs loaded simultaneously
   iii) Narrow-based tower foundation with two footings loaded in compression and two footings loaded in uplift simultaneously.

This report summarizes the results of the uplift testing program and documents their implications on the design and construction of drilled pier foundations for narrow based lattice transmission towers.

SOIL PROPERTIES

The characterization of the soil properties was made from samples retrieved from the SPT split spoon and from observation of the augered soil between sampling depths at the locations shown in Figure 1. The unconfined compression shear strength data summarized in Figure 2 are shown in more detail in Appendix A. Generally, the measured shear strength of the soil increases with depth as does the SPT blow count. The upper layer was a sandy silt till which extended to approximately 4.5 m. Underlying this was a water bearing sand layer extending to a depth of approximately 5.0 m. Beneath this was a sandy silt till. Borehole termination was at 5.6 m. The boreholes caved at the depth of the sand layer and the water level was at 3.5 m immediately after drilling which rose to 1.0 m on the following day.

The ground surface was relatively flat, a 1% slope provided surface drainage away from the test area. One augered hole at the location indicated in Figure 1 had to be abandoned because the auger intersected a boulder in the sandy silt till. The size of the boulder could not be determined but it was larger than the size that can be removed with one of the auger attachments. Soil testing at this location did not reveal contact with boulders. Footing 3 was terminated at a depth of 5.2 m because of an intersection with the edge of a large boulder.

TEST FOUNDATION

Size

All the footings were of 0.97 m diameter and lengths varying from 5.2 m to 5.6 m as indicated in Table I. Using a steel liner for temporary support required the hole size to be increased to 0.97 m from the proposed 0.92 m. Spacing between footings, center to center, was maintained at 1.68 m as planned, resulting in a spacing to diameter ratio (S/B) of 1.73 rather than 1.83 as in the narrow base footing design. The surface of all footings were flush with the ground surface.

Excavation and Footing Construction

Excavation of the holes was accomplished with a Watson 3000 caisson drill fitted with an auger. The hole was excavated to a depth of 4.5 m using a 0.92 m diameter auger. The auger was removed and back spun away from the hole at approximately 1.0 m intervals of advancement. When the hole was advanced to a depth of 4.5 m, which is just above the water bearing sand layer, the hole was enlarged to a diameter of 0.97 m by attaching an extended tooth to the auger flight. This facilitated the insertion of the 0.92 m diameter steel liner. The liner was then pushed through the water bearing sand layer into the sandy silt beneath. The hole was advanced to the final depth of 5.6 m using the 0.92 m diameter auger.

Materials

All the footings incorporated a conventional cage of reinforcing steel over the full depth of the footing. The concrete was specified to attain a compressive strength of 30 MPa in 28 days using type 10 cement and 20 mm maximum diameter aggregate. The superplasticizer Mighty 150 and a half dosage of PD25XL were used to obtain good flow characteristics without segregation during the continuous pouring of each footing. A pair of 57 mm Dywidag threaded steel rods 3.0 m long were cast in the centre of each footing as pulling bars. The pulling bars had pieces of reinforcing steel welded to them to maintain the correct spacing and to facilitate their handling as one unit. The compression footings had a coupler attached to the top of each Dywidag rod and was positioned 50 mm below grade so that the footing could be tested in uplift following compression loading. The holes for two footings could be augered without having to move the auger, thus ensuring that the holes would be parallel to each other over their full length.

The steel liner remained in the hole while the concrete was tremied up to the 3.0 m depth, at which point the liner was removed. The steel reinforcing cage was then lowered into position and pushed through the fluid concrete to rest on the bottom of the hole. Concrete was added up to ground level. The pulling bars were inserted and correctly positioned using a tape and transit. Straw was then placed over the concrete to prevent the fresh concrete from freezing. Each footing was completed prior to the augering of an adjacent hole.

Load Testing

Load testing was carried out by the Mechanical Research Department on eight footings.

The load testing sequence was as follows: Footings 5 and 6 were loaded simultaneously in uplift; Footings 1 and 2 were loaded in uplift and at the same time Footings 3 and 4 were loaded in compression; Footings 3 and 4 were then loaded in uplift; Footing 8 was loaded in uplift; Footing 7 was loaded in uplift. Table II describes the tests and gives the measured uplift capacities of the test footings.

The loading procedure was to load the footing in 100 kN increments with a hold time of 1 minute before additional load was applied. At higher load levels, the hold time was increased to allow the rate of footing creep to be reduced prior to the addition of further load.

Schematics, descriptions and photographs of the load testing equipment used, are presented in Appendix B.

Data Collection

The load-deflection data was collected using a Campbell Scientific 21X data logger. The load was measured from the load cell and the deflection of the footing was recorded by two LVDT's (Linear Voltage Displacement Transformers) per footing. Figures 3 to 6 show the load versus deflection curves for each test.

RESULTS

Constructability

The footings constructed were about one half scale of the proposed narrow base tower foundation. The spacing to diameter ratio was slightly less than the prototype spacing to diameter ratio of 1.83 because of the increased diameter required to use the steel liner. The four closely spaced footings were constructed without complication. Out of a total of eight footings constructed and tested, two had to be relocated because of intersecting a boulder and the hole caved during construction. The caved hole was the first trial hole augered without temporary casing to see if it would remain open for the duration it would take to get the concrete placed to above the depth of the sand seam. The hole did not stay open and a steel liner for temporary excavation support was used to avoid this problem. Both of these problems should be considered when constructing

closely spaced drilled piers for narrow-based towers in glacial tills.

Ultimate Capacity

The ultimate uplift resistance of a single pile or pier in sand is:

$$Qu = 1/2\ \pi B\ D^2\ \gamma Ku \tan \emptyset + W \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

where B = Pile diameter
D = Embedment Length
γ = Unit weight of soil
Ku = Empirical uplift coefficient/1/
$\emptyset$ = Friction angle
W = Self weight of the pile

The value for Ku back calculated from field and laboratory tests with the footings spaced far apart has been related to the depth to diameter ratio and the friction angle of the soil/1,4,5,6/. Figure 7 shows a design chart developed using statistical methods as well as theoretical values of the uplift coefficient Ku in sands, for the range of depth to diameter (D/B) ratios normally encountered in transmission tower design.

The ultimate uplift resistance of a single pile or pier in clay is:

$$Qu = \pi B\ Ca\ D + W \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

where B = Pile diameter
Ca = α Cu
α = Adhesion factor
Cu = Undrained cohesion
D = Embedment length
W = Self weight of the pile

The variation of the adhesion factor with undrained cohesion is shown in Figure 8. This figure was developed using statistical methods on published data/8/. Table III summarizes the test results and backcalculated Ku values.

Load Deflection Behaviour

The load deflection behaviour of Footings 7 and 8, which were loaded in uplift as single footings, are shown in Figures 2 to 4 and summarized in Table II. The loading of Footing 8 was halted at 2290 kN because of the large amount of deflection in the beams. This was caused by a greater distance between the steel reaction pads than that which was used during the testing of Footing 7. A complete description of the mechanical aspects of the testing procedure is included in Appendix B. The load deflection behaviour of Footing 8 shown in Figure 3 agrees very closely up to the 1600 kN load, thus the ultimate capacity of Footing 8 could be extrapolated beyond 2290 kN to an expected value which would be closer to although somewhat less than the ultimate capacity of 2757 kN for Footing 7. Therefore, the capacity measured for Footing 7 is taken as the reference capacity of a single footing.

The theoretical capacity of 2803 kN, shown in Table II, obtained by using an uplift coefficient Ku of 3 in equation (1), agrees to within 2% of the measured reference capacity of 2757 kN. Using a cohesive analysis the predicted capacity was 2105 kN, which is 24% lower than the measured uplift capacity. Therefore, the best prediction of uplift capacity in this sandy silt till is obtained by the cohesionless analysis.

The load deflection response of the single Footings 7 and 8, shown in Figure 3, remains linear up to approximately 60% of the ultimate capacity, at which point the change in deflection increases with every step of loading towards failure. Examination of the load deflection behaviour shown in Figures 4 to 6 for footing pairs in uplift show a rounded curve without an initial linear portion. This is most noted for footing pairs 3, 4 and 1, 2, which were part of the 4 footing tests.

Similar ultimate capacities were reached by the three pairs of footings tested but the amount of deflection which could be expected at working loads (say 50% of ultimate) was greater for the four footing arrangement than was recorded for the isolated pair 5, 6. This difference could possibly be explained by an altered shear zone around the uplift pair of footings, due to the presence of another pair of footings.

There was no difference in the behaviour of pair 1, 2 during the testing with the simultaneous loading of pair 3,4 in compression and with that from the subsequent testing of pairs 3,4 in uplift. It thus appears that the compression loading does not have a detrimental effect on the uplift behaviour of the narrow base tower footings.

Group Effect

A reduction in the uplift capacity of closely spaced footing pairs relative to the capacity of a single footing was expected because, for footing pairs, the shear zone around each footing, which provides the uplift resistance, overlaps. The contribution to uplift resistance in the overlapping zone must be shared between the footings. From the uplift failure load and the footing dimensions it is possible to back calculate for Ku using equation (1). These back calculated Ku values are shown in Table II. The benefit of comparing Ku values rather than capacities is that meaningful comparisons between different sized footings in cohesionless soils with differing properties can be made.

The group reduction factor is expected to be larger for footing pairs with lower S/B ratios and with increased footing depth because of the greater amount of shear zone overlap. These trends can be seen in the group reduction factors shown in Table III which were obtained from this test and a previous test. The back calculated Ku value of 2.95 for single footings is in agreement with the Ku = 3.0 recommended for footings of this D/B ratio constructed in dense sands/1,6/. It should be noted that in the current design practice a value of 2.0 is used in dense sands based on old field test data on footings with D/B ratios of only 3 to 4/9/. The Ku value from all the footing pairs tested averaged 1.89 +/- 0.03. Based on the results obtained at this site for the D/B and S/B ratios used, a group capacity reduction factor of 36% should be applied to the single

footing capacity. Similar reduction factor was obtained from earlier tests in compact sand at Essa TS/2/ (Table III).

SUMMARY

This field testing program has shown that closely spaced large diameter augered footings can be constructed in sandy silt till for the narrow base towers. Temporary steel liners will be needed to prevent sides from caving below the water table.

The load deflection behaviour of eight test footings loaded in uplift were measured. The cohesionless analysis provided the best estimate of the uplift capacity. Back calculation of the uplift coefficient Ku from the test data revealed the following:

i) single footing Ku = 2.95
ii) footing pairs Ku = 1.89 +/- 0.03 (3 tests)

Similar values were obtained from load tests in compact sand at Essa. In addition, the simultaneous loading of two of the four footings in compression showed no detrimental effect on the uplift capacity of the other two footings. Slightly more deflection occurred at lower load levels for footings from the four footing configuration than for either the isolated pair of footings or the single footings. Based on the results of load tests at this site, for the footing sizes and spacings used, a group capacity reduction factor of 36% should be applied to the single footing capacity.

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TABLE I

Footing Dimensions as Constructed

Footing	Depth (D) (m)	Diameter (B) (m)
1	5.49	0.97
2	5.59	0.97
3	5.18	0.97
4	5.64	0.97
5	5.59	0.97
6	5.59	0.97
7	5.59	0.97
8	5.59	0.97

TABLE II

Load-Deflection Behaviour

Theoretical Values

Footing

	Cohesionless Analysis		Cohesive Analysis	
	Ku	Ultimate Capacity	Ca	Ultimate Capacity
7 or 8	3.0	2803 kN	118 kPa	2105 kN

Measured Value

Footing	Uplift Loading Method	Estimated Self Weight (kN)	Measured Ultimate Capacity (kN)	Back calculated Ku
7	Single footing	95	2757	2.95
8	Single footing	95	2290*	2.43*
5	Loaded as a pair	95	1824	1.91
6	Simultaneously	95	1779	1.87
1++	Loaded as a pair	93	1766	1.85
2	with 3 and 4 in compression	95	1833	1.92
3	Loaded as a pair	88	1801	1.97
4	after compression	96	1801	1.87

For footing pairs, average Ku = 1.89 +/- 0.03

Estimated reduction due to group effect 2.95-1.89/2.95 = 36%

*Not the Ultimate Capacity
++Simulated the loading from a narrow based compact tower

TABLE III

Summary of Group Capacity Reduction Factors

B	D	D/B	S	S/B	Ku Single	Ku Group	Group Reduction in Ku	Soil/Referenced
0.96 m	5.59 m	5.8	1.67 m	1.73	2.95	1.89 +/- 0.03	36%	sandy, silt till
0.71 m	3.66 m	5.2	2.36 m	1.92	2.83 +/- 0.25	1.85	35%	compact sand/2/
0.71 m	3.66 m	5.2	1.74 m	2.45	2.83 +/- 0.25	2.0	29%	compact sand/2/

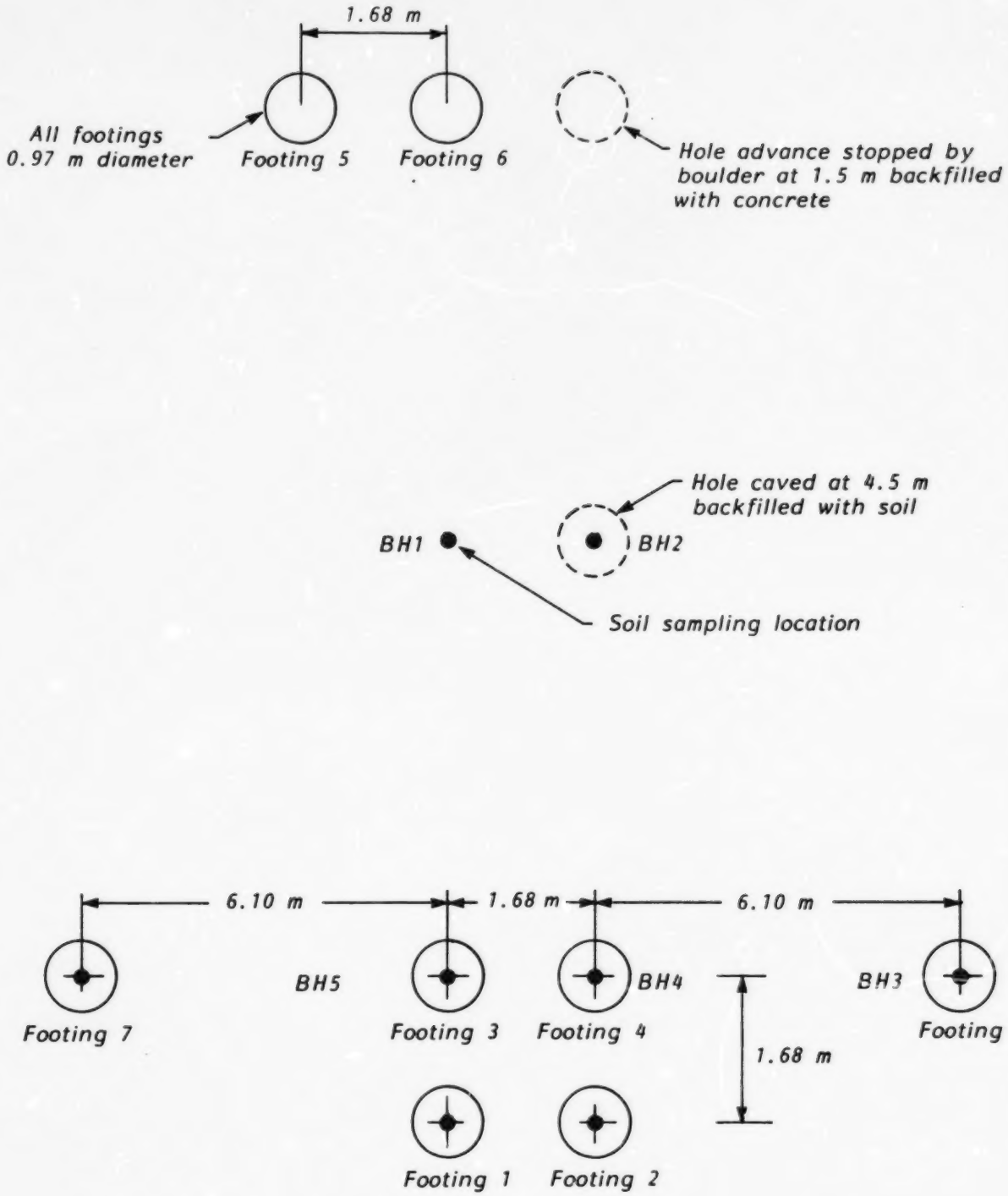

FIGURE 1
FOOTING LOCATIONS AT TEST SITE

FIGURE 2 SOIL STRENGTH FROM UNCONFINED COMPRESSION TESTS VS. SAMPLE DEPTH

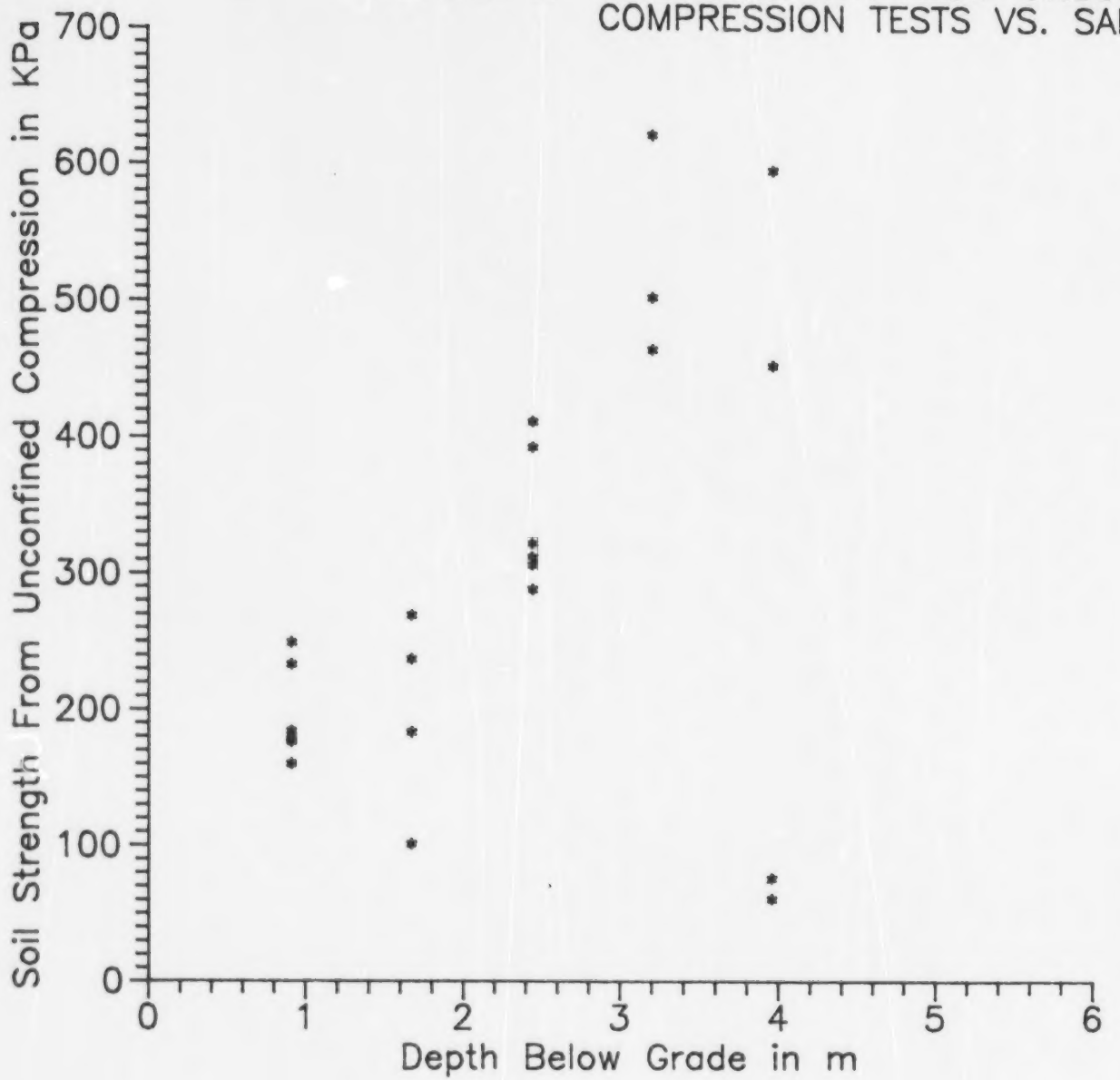

FIGURE 3 DEFLECTION OF FOOTINGS 7 AND 8
(BOTH WERE TESTED AS SINGLE FOOTINGS)

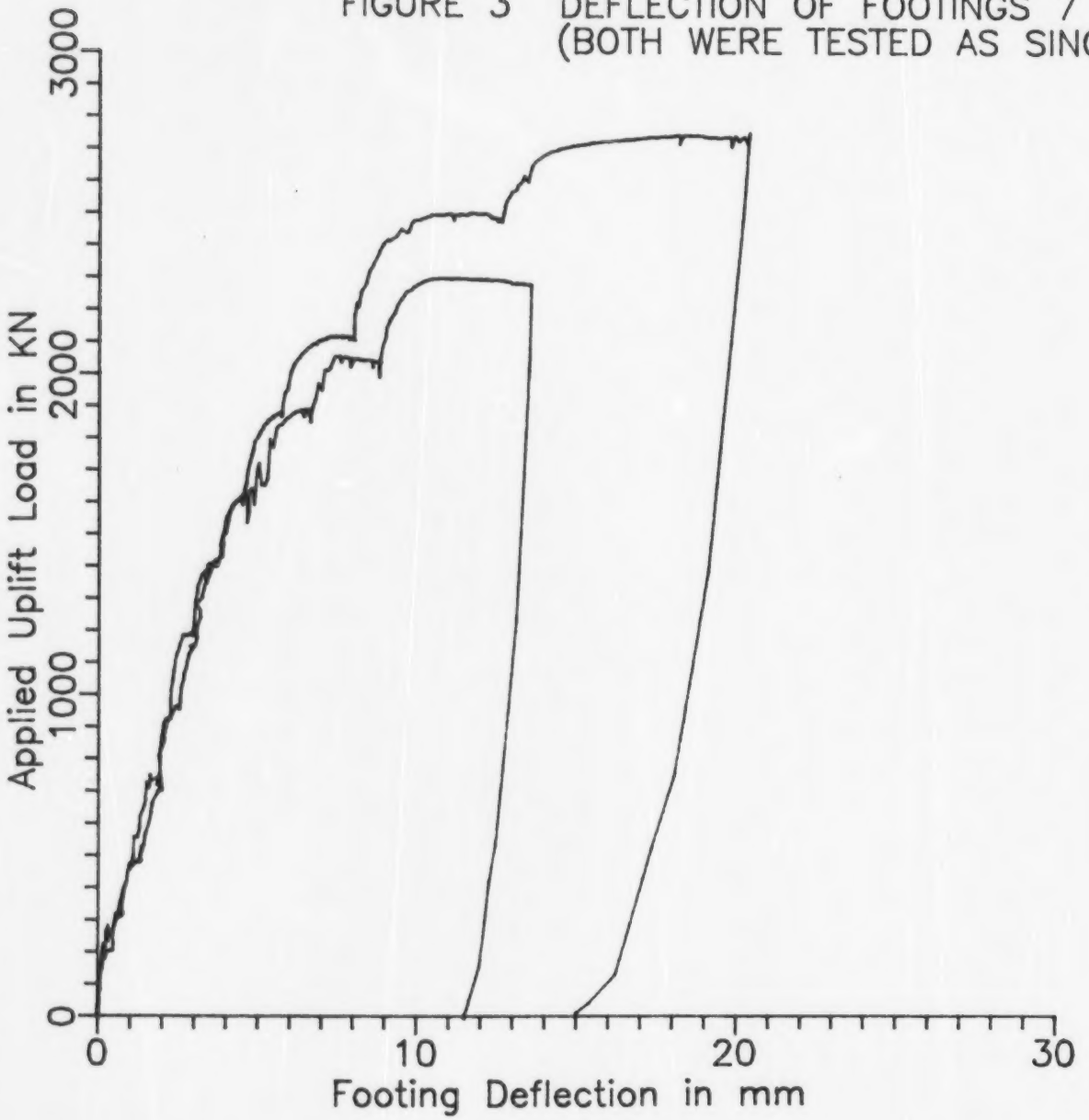

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FIGURE 4 DEFLECTION OF FOOTINGS 5 AND 6
(PULLED AS A PAIR)

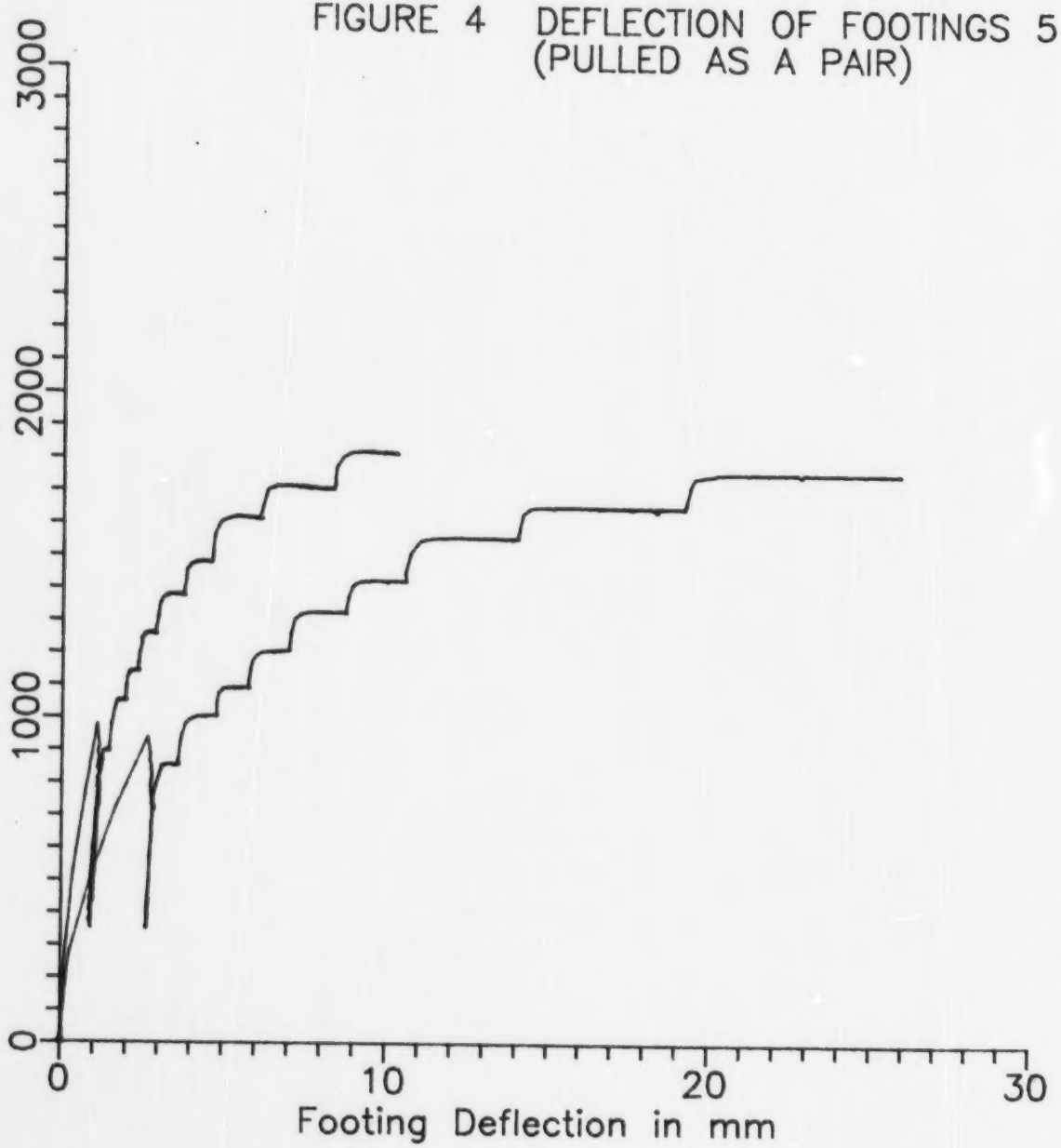


FIGURE 5 DEFLECTION OF FOOTINGS 1 AND 2 DURING THE FOUR FOOTING TEST

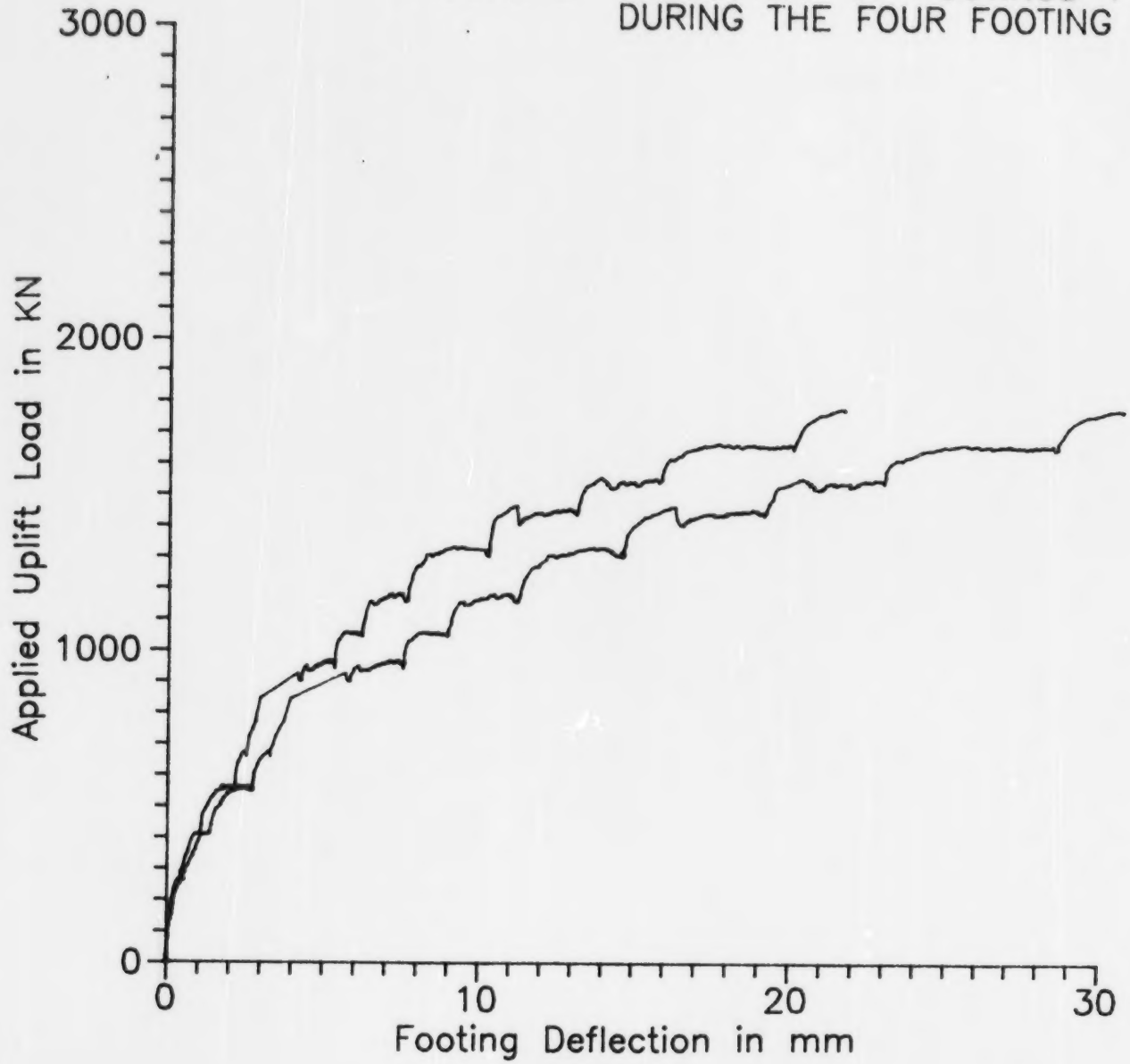
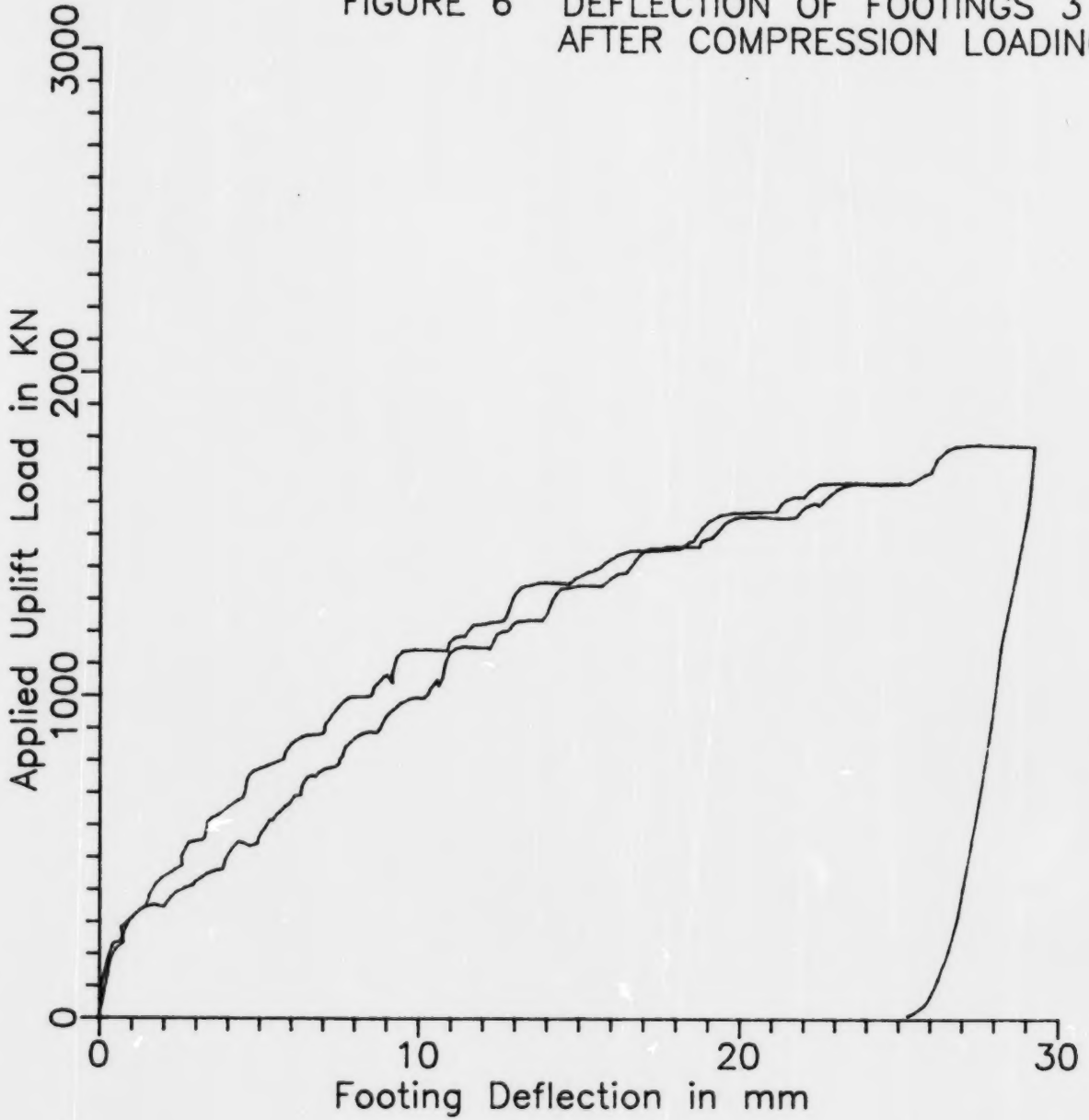


FIGURE 6 DEFLECTION OF FOOTINGS 3 AND 4 (PULLED AS AFTER COMPRESSION LOADING



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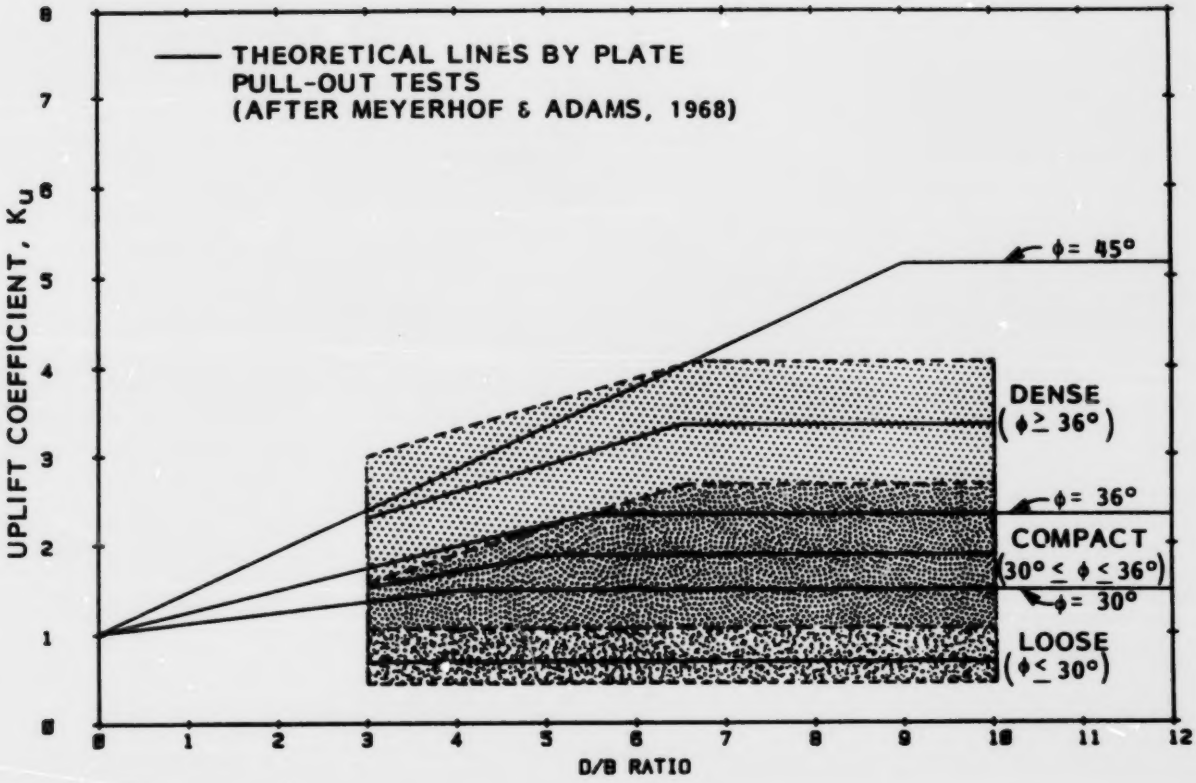

FIGURE 7

COMPARISON OF COMBINED STATISTICAL UPLIFT COEFFICIENT BANDS WITH Ku THEORY

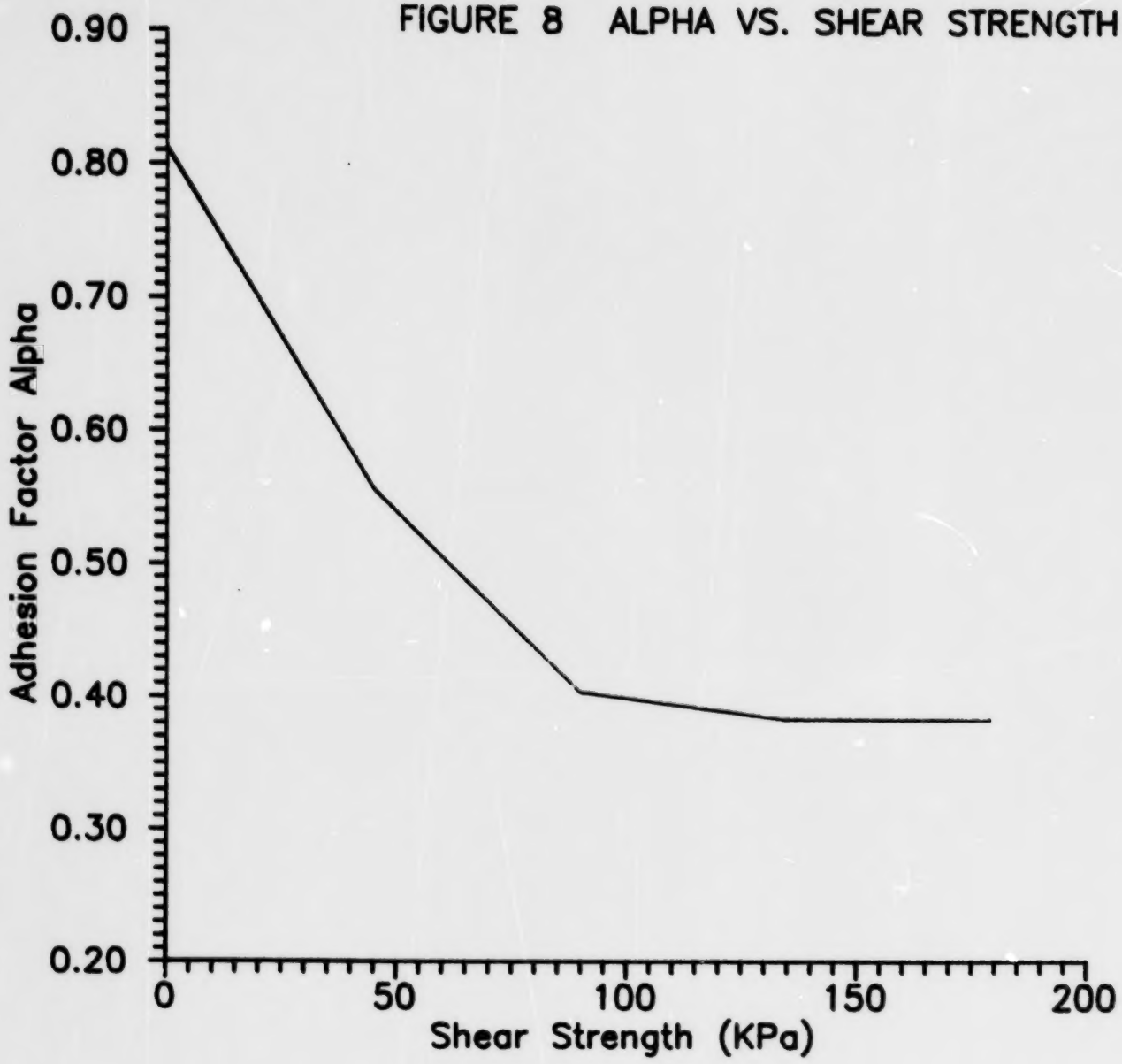

FIGURE 8 ALPHA VS. SHEAR STRENGTH

APPENDIX A

SITE CONDITIONS AND SOIL PROPERTIES

PROJECT MILTON TEST FOUNDATIONS — HOLE TYPE FLYTE AUGER — OBSERVER DF

LOCATION FOOTING 1 — HOLE SIZE — COMPILED PST

DIP — DRILLED FEB 26, 1987 — CHECKED

DATUM — BEARING — DRAWN — APPROVED

BORING DATA						BORING LOG	WATER INF.		FIELD TESTS		LABORATORY TESTS					
SCALE - METRES	ELEV / DEPTH	(1) SAMPLE NO	(2) SAMPLE TYPE	(3) 'N' VALUE per 0.3m / ROD - %	(4) WATER LEVEL and CAVING / W.R	(5) DESCRIPTION / (6) REMARKS	(7) OBSERVATION WELL DATA	(8) 'k' soil/rock cm/s / WATER PRESS TEST	(9) SPECIAL TESTS TYPE	(9) SPECIAL TESTS RESULTS	NATURAL DENSITY kg/m³	MOISTURE CONTENT - %	(10) SHEAR STRENGTH kPa TYPE	(10) SHEAR STRENGTH kPa RESULTS	(11) SPECIAL TESTS TYPE	(11) SPECIAL TESTS RESULTS
1		A1		34		*Sandy Silt Till, dense to very dense, little clay and fine gravel, brown, moist*					2201	15.1	UC	784		
2		A2		53							2324	12.0				
		A3		52		*Change to grey at 3.0 m*					2289	11.8				
3		A4		67							2370	14.2				
4		A5		68	▽						2338	10.2	UC	61		
					C	4.5 m *Sand very dense, fine, grey, wet*						14.6				
5		A6		57		5.3 m										
		A7		60/.07		*Sandy Silt Till, very dense, red-brown, shale and gravel pieces, moist* 5.5 m					2345	7.0				
6						*End of Borehole*										

GEOTECHNICAL DATA SHEET

hole no 7 — Fig 1A

sheet 1 OF 1

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PROJECT MILTON TEST FOUNDATIONS — HOLE TYPE FLYTE AUGER — OBSERVER DF

LOCATION FOOTING 2 — HOLE SIZE — COMPILED PST

DIP — DRILLED FEB 26, 1987 — CHECKED

DATUM — BEARING — DRAWN — APPROVED

SCALE - METRES	ELEV / DEPTH	(1) SAMPLE NO	(2) SAMPLE TYPE	(3) 'N' VALUE per 0.3m / ROD - %	(4) WATER LEVEL and CAVING / W.R.	BORING LOG (5) DESCRIPTION	(6) REMARKS	WATER INF. (7) OBSERVATION WELL DATA	(8) 'k' soil/rock cm/s / WATER PRESS TEST	FIELD TESTS (9) SPECIAL TESTS TYPE	RESULTS	LABORATORY TESTS: NATURAL DENSITY kg/m³	MOISTURE CONTENT - %	(10) SHEAR STRENGTH kPa TYPE	RESULTS	(11) SPECIAL TESTS TYPE	RESULTS
	1	A1		32		Sandy Silt Till, dense, to very dense, little clay and fine gravel, brown, moist						2151	12.7	UC	233		
	2	A2		48								2304	11.4	UC	237		
		A3		49								2284	12.4	UC	393		
	3	A4		66	▽								11.9				
						Change to grey at 3.8 m											
	4	A5		115								2334	10.2	UC	76		
						4.6 m											
		A6		85	C	Sand, very dense, fine, grey, wet							9.7				
	5					5.2 m											
		A7		85/.22		Sandy Silt Till, very dense, red-brown, shale and gravel pieces, moist 5.6 m						2362	9.2				
	6					End of Borehole											

GEOTECHNICAL DATA SHEET

hole no 8 — Fig 2A

sheet 1 OF 1

dwg no 2881B-RD

PROJECT MILTON TEST FOUNDATIONS — HOLE TYPE FLYTE AUGER — OBSERVER DF

LOCATION FOOTING 3 — HOLE SIZE — COMPILED PST

DIP — DRILLED FEB 26, 1987 — CHECKED

DATUM — BEARING — DRAWN — APPROVED

SCALE-METRES	ELEV / DEPTH	(1) SAMPLE NO	(2) SAMPLE TYPE	(3) 'N' VALUE per 0.3m / RQD-%	(4) WATER LEVEL and CAVING W.L.	(5) DESCRIPTION	(6) REMARKS	(7) OBSERVATION WELL DATA	(8) 'k' cm/s WATER PRESS TEST	FIELD TESTS (9) SPECIAL TESTS TYPE	FIELD TESTS (9) SPECIAL TESTS RESULTS	NATURAL DENSITY kg/m³	MOISTURE CONTENT - %	LABORATORY TESTS (10) SHEAR STRENGTH kPa TYPE	(10) SHEAR STRENGTH kPa RESULTS	(11) SPECIAL TESTS TYPE	(11) SPECIAL TESTS RESULTS
1		A1		30		Sandy Silt Till, dense to very dense, little clay and fine gravel, brown, moist						2180	16.8	UC	249		
		A2		50								2298	11.6				
2					▽												
		A3		41		Change to grey at 2.9 m						2289	10.6	UC	283		
3																	
		A4		37								2252	12.7				
4		A5		95		4.3 m											
					C	Sand, very dense, fine, grey, wet						2324	8.2				
		A6		62													
5						5.1 m							15.3				
		A7		92/.22		Sandy Silt Till, very dense, red-brown, shale and gravel pieces, moist							7.4				
						5.6 m											
6						End of Borehole											

GEOTECHNICAL DATA SHEET

hole no 5 — Fig 3A

sheet 1 OF 1

dwg no 200811-RD

PROJECT MILTON TEST FOUNDATIONS — HOLE TYPE FLYTE AUGER — OBSERVER DF

LOCATION FOOTING 4 — HOLE SIZE — COMPILED PST

DIP — DRILLED FEB 25, 1987 — CHECKED

DATUM — BEARING — DRAWN — APPROVED

BORING DATA					BORING LOG	WATER INF.		FIELD TESTS		LABORATORY TESTS					
SCALE - METRES	ELEV / DEPTH	(1) SAMPLE NO	(2) SAMPLE TYPE	(3) 'N' VALUE per 0.3m / RQD - %	(4) WATER LEVEL and CAVING / W.R	(5) DESCRIPTION / (6) REMARKS	(7) OBSERVATION WELL DATA	(8) 'k' cm/s / WATER PRESS TEST	(9) SPECIAL TESTS TYPE	(9) SPECIAL TESTS RESULTS	NATURAL DENSITY kg/m³	MOISTURE CONTENT - %	(10) SHEAR STRENGTH kPa TYPE	(10) SHEAR STRENGTH kPa RESULTS	(11) SPECIAL TESTS TYPE	(11) SPECIAL TESTS RESULTS
1		A1		58		Sandy Silt Till, dense to very dense little clay and fine gravel, brown, moist					2164	16.7	UC	160		
		A2		59							2302	11.5	UC	183		
2		A3		45							2284	10.9	UC	312		
3		A4		62	▽	Change to grey at 3.6 m					2377	11.1				
4		A5		77/.30							2389	10.1				
					C	4.5 m										
		A6		42		Sand, very dense, fine, grey, wet						16.8				
5						5.0 m										
		A7		112/.22		Sandy Silt Till, very dense, red-brown, shale and grave. pieces, moist					2402	7.8				
						5.6 m										
6						End of Borehole										

GEOTECHNICAL DATA SHEET

hole no 4 — Fig 4A

sheet 1 OF 1

dwg no 20813-RD

PROJECT MILTON TEST FOUNDATIONS — HOLE TYPE FLYTE AUGER — OBSERVER DF

LOCATION AS SHOWN IN FIGURE 1 — HOLE SIZE 100 m — COMPILED PST

DIP — DRILLED FEB 25, 1987 — CHECKED

DATUM — BEARING — DRAWN — APPROVED

SCALE - METRES	ELEV / DEPTH	(1) SAMPLE NO	(2) SAMPLE TYPE	(3) 'N' VALUE per 0.3m / RQD - %	(4) WATER LEVEL and CAVING / W.R	BORING LOG (5) DESCRIPTION	(6) REMARKS	WATER INF. (7) OBSERVATION WELL DATA	(8) 'k' cm/s soil/rock	WATER PRESS TEST	FIELD TESTS (9) SPECIAL TESTS TYPE	RESULTS	LABORATORY TESTS: NATURAL DENSITY kg/m³	MOISTURE CONTENT - %	(10) SHEAR STRENGTH kPa TYPE	RESULTS	(11) SPECIAL TESTS TYPE	RESULTS
	1	A1		31	▽	Sandy Silt Till, dense to very dense, little clay and fine gravel, brown, moist							2269	18.3				
		A2		46									2346	9.9	UC	269		
	2	A3		57									2274	11.3	UC	322		
	3	A4		58		Change to grey at 3.0 m							2354	8.5	UC	622		
	4	A5		80									2361	8.3	UC	453		
						4.5 m												
		A6		78	C	Sand, very dense, fine, grey, wet								14.8				
	5					5.1 m												
		A7		100/.16		Sandy Silt Till, very dense, red-brown, shale and gravel pieces, moist							2386	7.0				
						End of Borehole												
	6																	

GEOTECHNICAL DATA SHEET

hole no 1 — Fig 5A

sheet 1 OF 1

dwg no 20013-RD

87-134

PROJECT MILTON TEST FOUNDATIONS — HOLE TYPE FLYTE AUGER — OBSERVER DF

LOCATION AS SHOWN IN FIGURE 1 — HOLE SIZE — COMPILED PST

DIP — DRILLED FEB 25, 1987 — CHECKED

DATUM — BEARING — DRAWN — APPROVED

BORING DATA						BORING LOG		WATER INF.		FIELD TESTS		LABORATORY TESTS					
SCALE-METRES	ELEV / DEPTH	(1) SAMPLE NO	(2) SAMPLE TYPE	(3) 'N' VALUE per 0.3m / ROD-%	(4) WATER LEVEL and CAVING	(5) DESCRIPTION	(6) REMARKS	(7) OBSERVATION WELL DATA	(8) 'k' cm/s / WATER PRESS TEST	(9) SPECIAL TESTS TYPE	(9) SPECIAL TESTS RESULTS	NATURAL DENSITY kg/m³	MOISTURE CONTENT-%	(10) SHEAR STRENGTH kPa TYPE	(10) SHEAR STRENGTH kPa RESULTS	(11) SPECIAL TESTS TYPE	(11) SPECIAL TESTS RESULTS
1		A1		34	▽	*Sandy Silt Till, dense to very dense, little clay and fine gravel, brown, moist*						2116	18.1	UC	178		
		A2		54								2288	13.4	UC	101		
2		A3		90		*Change to grey at 2.9 m*						2317	10.7	UC	306		
3		A4		74								2346	8.5	UC	503		
4		A5		72		4.4 m							8.4				
		A6		120/.22	C	*Sand, very dense, fine, grey, wet* 4.9 m						2387	10.3				
5						*Sandy Silt Till, very dense, red-brown, shale and gravel pieces, moist*							12.8				
		A7		115/.22		5.6 m						2359	8.0				
6						*End of Borehole*											

dwg no 20881-RD

GEOTECHNICAL DATA SHEET — hole no 2 — Fig 6A — sheet 1 OF 1

87-134

PROJECT MILTON TEST FOUNDATIONS — HOLE TYPE FLYTE AUGER — OBSERVER DF

LOCATION FOOTING 7 — HOLE SIZE — COMPILED PST

DIP — DRILLED FEB 26, 1987 — CHECKED

DATUM — BEARING — DRAWN — APPROVED

BORING DATA						BORING LOG		WATER INF.		FIELD TESTS		LABORATORY TESTS					
SCALE-METRES	ELEV / DEPTH	(1) SAMPLE NO	(2) SAMPLE TYPE	(3) 'N' VALUE per 0.3m / RQD-%	(4) WATER LEVEL and CAVING W.R	(5) DESCRIPTION	(6) REMARKS	(7) OBSERVATION WELL DATA	(8) 'k' soil/rock cm/s / WATER PRESS. TEST	(9) SPECIAL TESTS TYPE	RESULTS	NATURAL DENSITY kg/m³	MOISTURE CONTENT - %	(10) SHEAR STRENGTH kPa TYPE	RESULTS	(11) SPECIAL TESTS TYPE	RESULTS
1		A1		35		*Sandy Silt Till, dense to very dense, little clay and fine gravel, brown, moist*						2200	15.6				
		A2		44								2324	11.6				
2		A3		56								2276	11.3	UC	412		
					▽	*Change to grey at 3.0 m*											
3		A4		70								2310	10.6	UC	596		
4		A5		76		*4.3 m*						2373	11.7				
					C	*Sand, very dense, fine, grey, wet*											
		A6		120		*4.9 m*											
5						*Sandy Silt Till, very dense, red-brown, shale and gravel pieces, moist*							15.8					
		A7		110/.20		*5.6 m*						2359	8.0				
						End of Borehole											
6																	

GEOTECHNICAL DATA SHEET — hole no 6 — Fig 7A — sheet 1 OF 1

dwg no 2008S1-RD

87-134

PROJECT MILTON TEST FOUNDATIONS — HOLE TYPE FLYTE AUGER — OBSERVER DF

LOCATION FOOTING 8 — HOLE SIZE — COMPILED PST

DIP — DRILLED FEB 25, 1987 — CHECKED

DATUM — BEARING — DRAWN — APPROVED

BORING DATA						BORING LOG		WATER INF.		FIELD TESTS		LABORATORY TESTS					
SCALE - METRES	ELEV / DEPTH	(1) SAMPLE NO	(2) SAMPLE TYPE	(3) 'N' VALUE per 0.3m / ROD - %	(4) WATER LEVEL and CAVING / W.R.	(5) DESCRIPTION	(6) REMARKS	(7) OBSERVATION WELL DATA	(8) 'k' cm/s / WATER PRESS TEST	(9) SPECIAL TESTS TYPE	(9) SPECIAL TESTS RESULTS	NATURAL DENSITY kg/m³	MOISTURE CONTENT - %	(10) SHEAR STRENGTH kPa TYPE	(10) SHEAR STRENGTH kPa RESULTS	(11) SPECIAL TESTS TYPE	(11) SPECIAL TESTS RESULTS
1		A1		27		*Sandy Silt Till, dense to very dense, little clay and fine gravel, brown, moist*						2132	18.3	UC	176		
		A2		39								2387	12.7				
2		A3		47	▽							2305	11.8				
3		A4		67								2330	9.9	UC	465		
4		A5		80/.30								2386	8.3				
					C	4.5 m											
		A6		67		*Sand, very dense, fine, grey, wet*							17.2				
5						5.1 m											
		A7		115/.22		*Sandy Silt Till, very dense, red-brown, shale and gravel pieces, moist* 5.6 m							6.9				
6						*End of Borehole*											

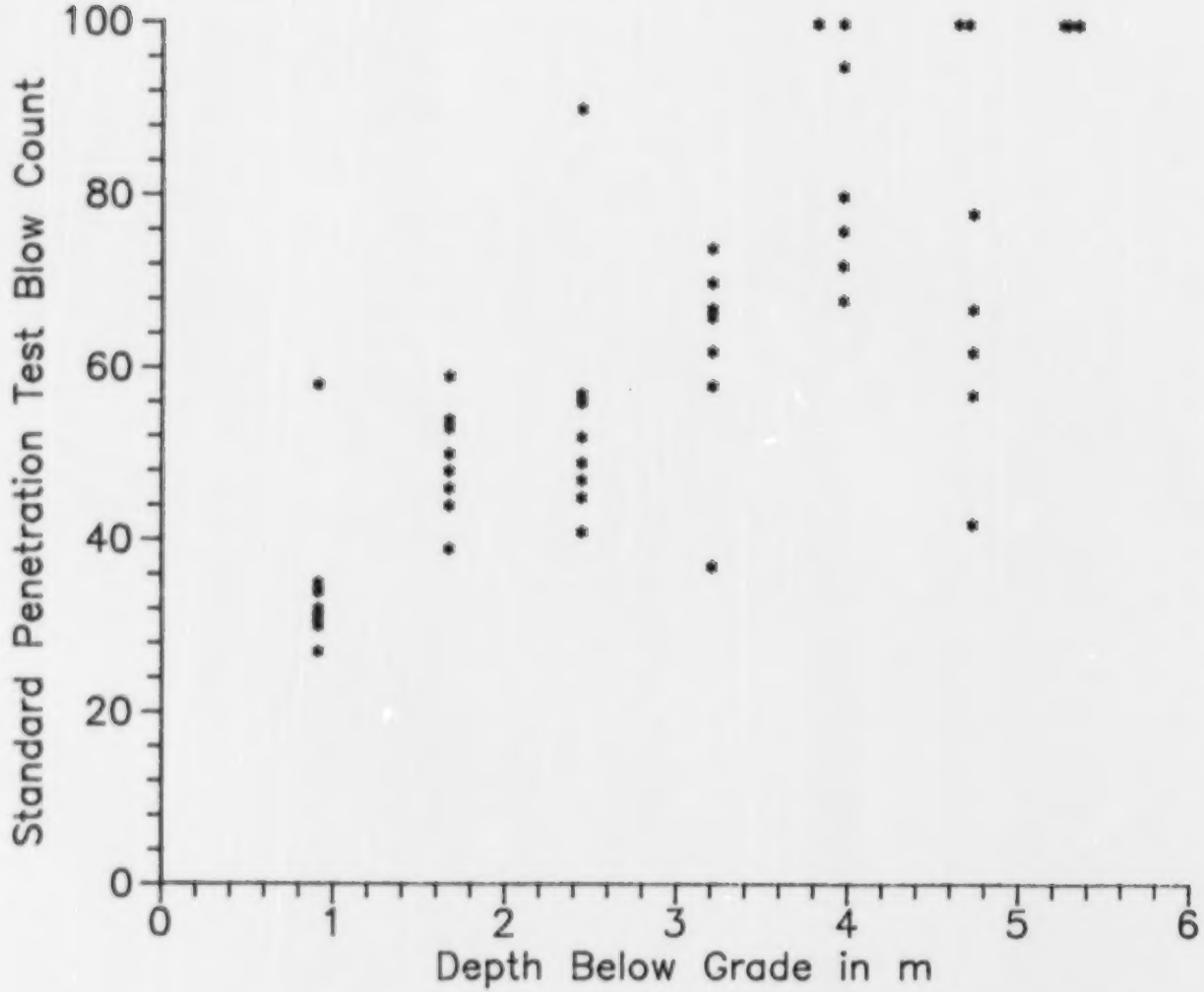
GEOTECHNICAL DATA SHEET

hole no 3 — Fig 8A

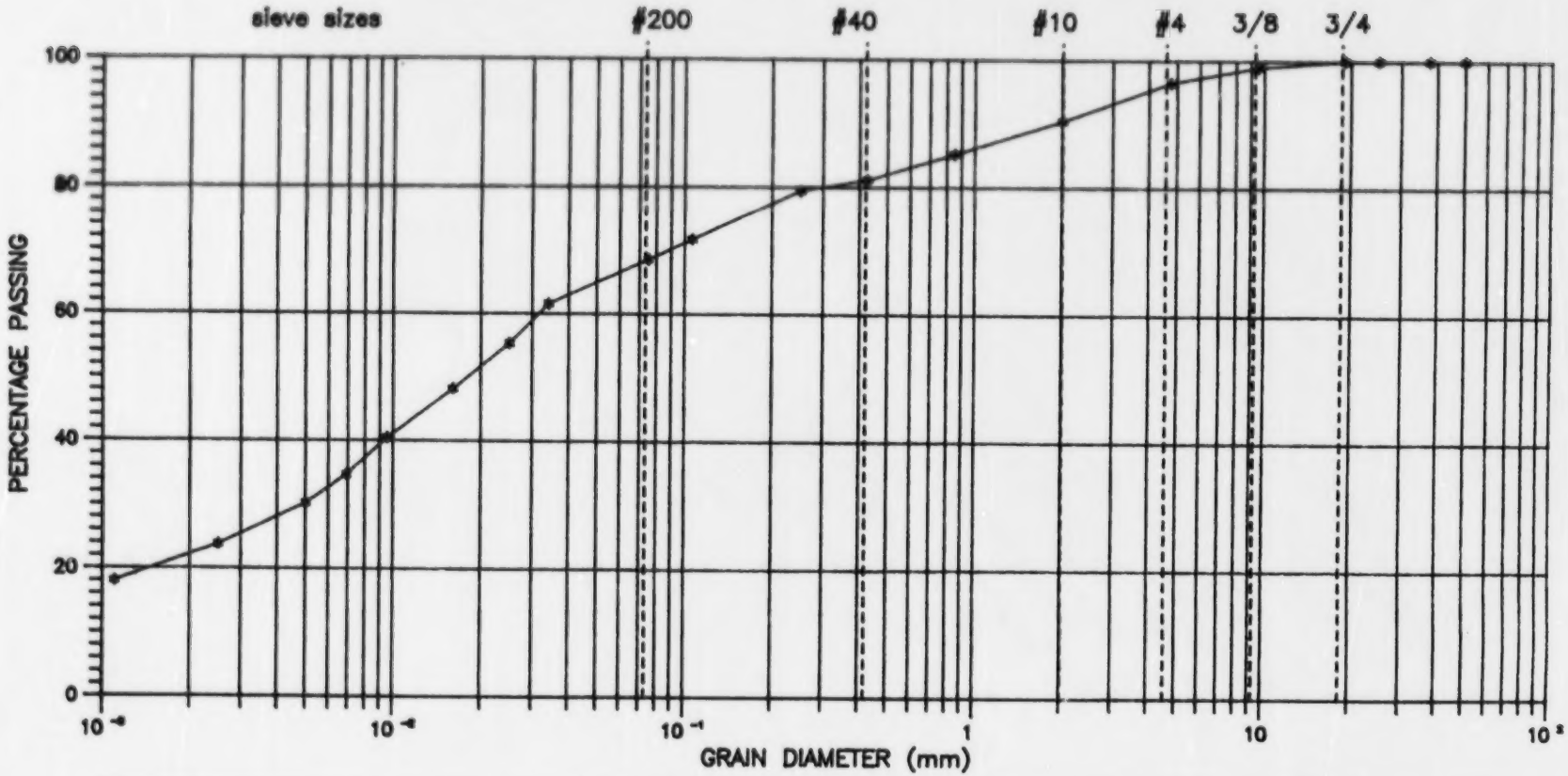
sheet 1 OF 1

dwg no 200816-RD

FIGURE 9A STANDARD PENETRATION TEST BLOW COUNT VS. DEPTH AT SAMPLING LOCATIONS



MECHANICAL ANALYSIS – MILTON T.S. BH–2 S–5



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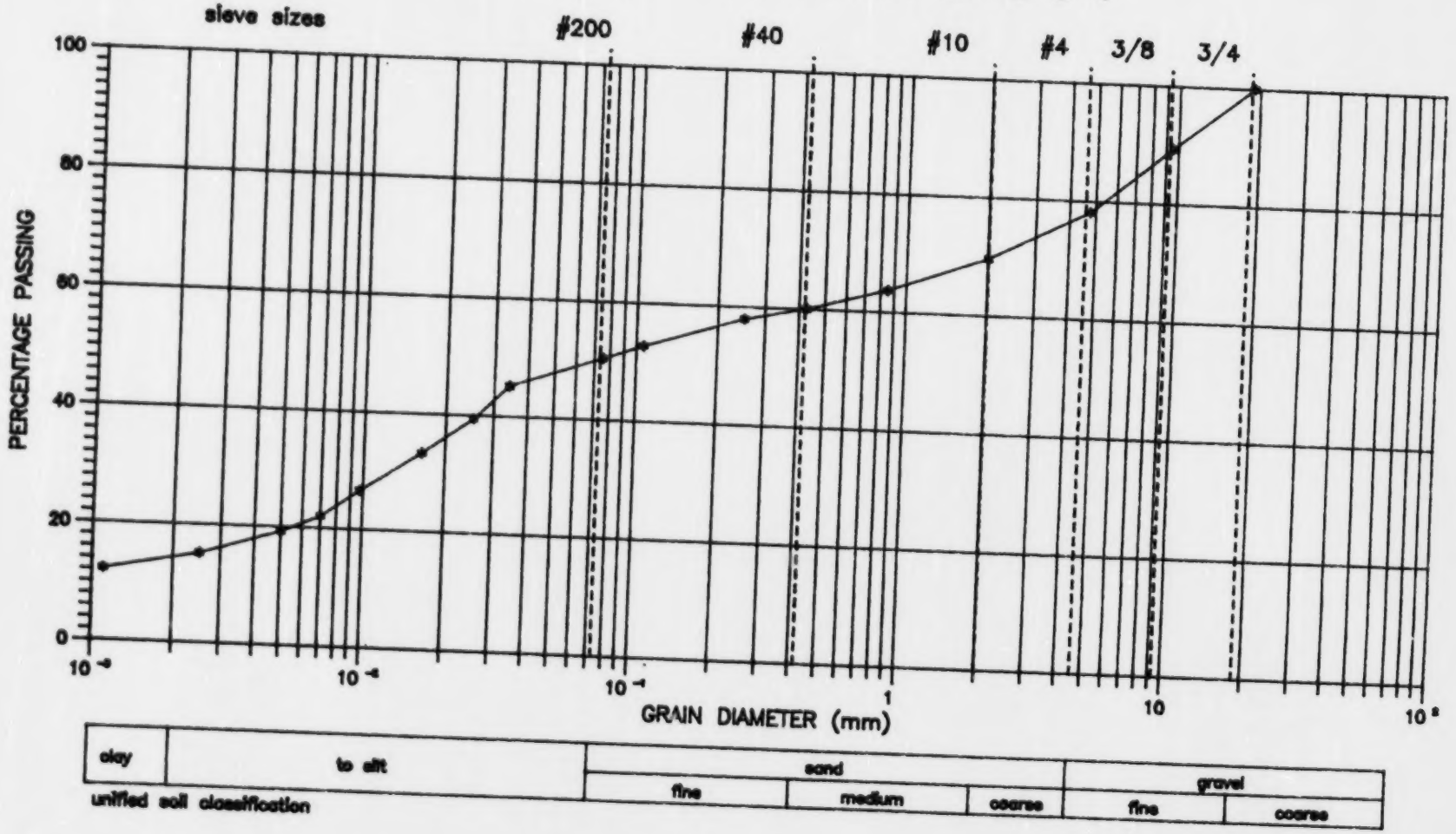
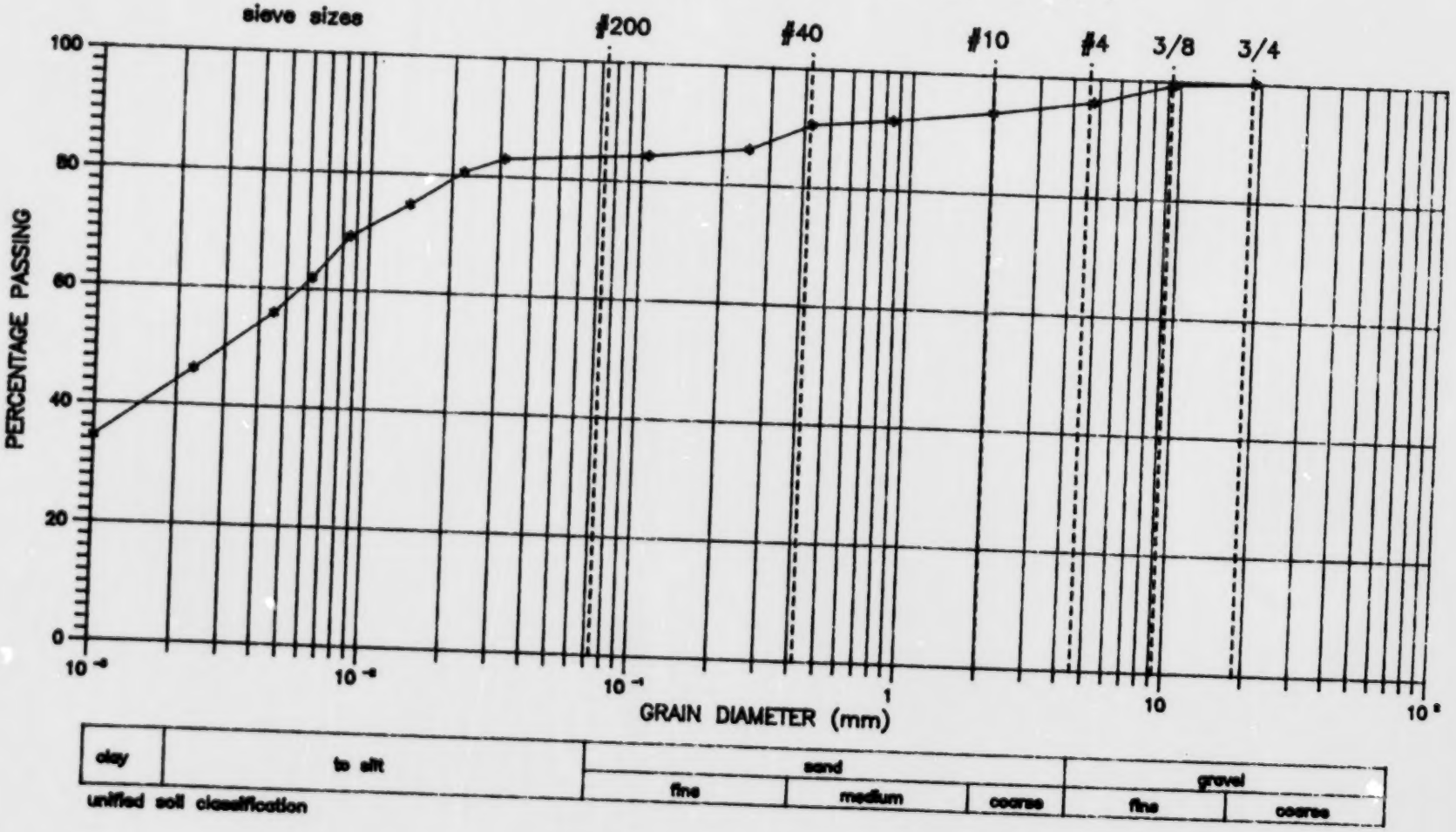
MECHANICAL ANALYSIS – MILTON T.S. BH–2 S–7



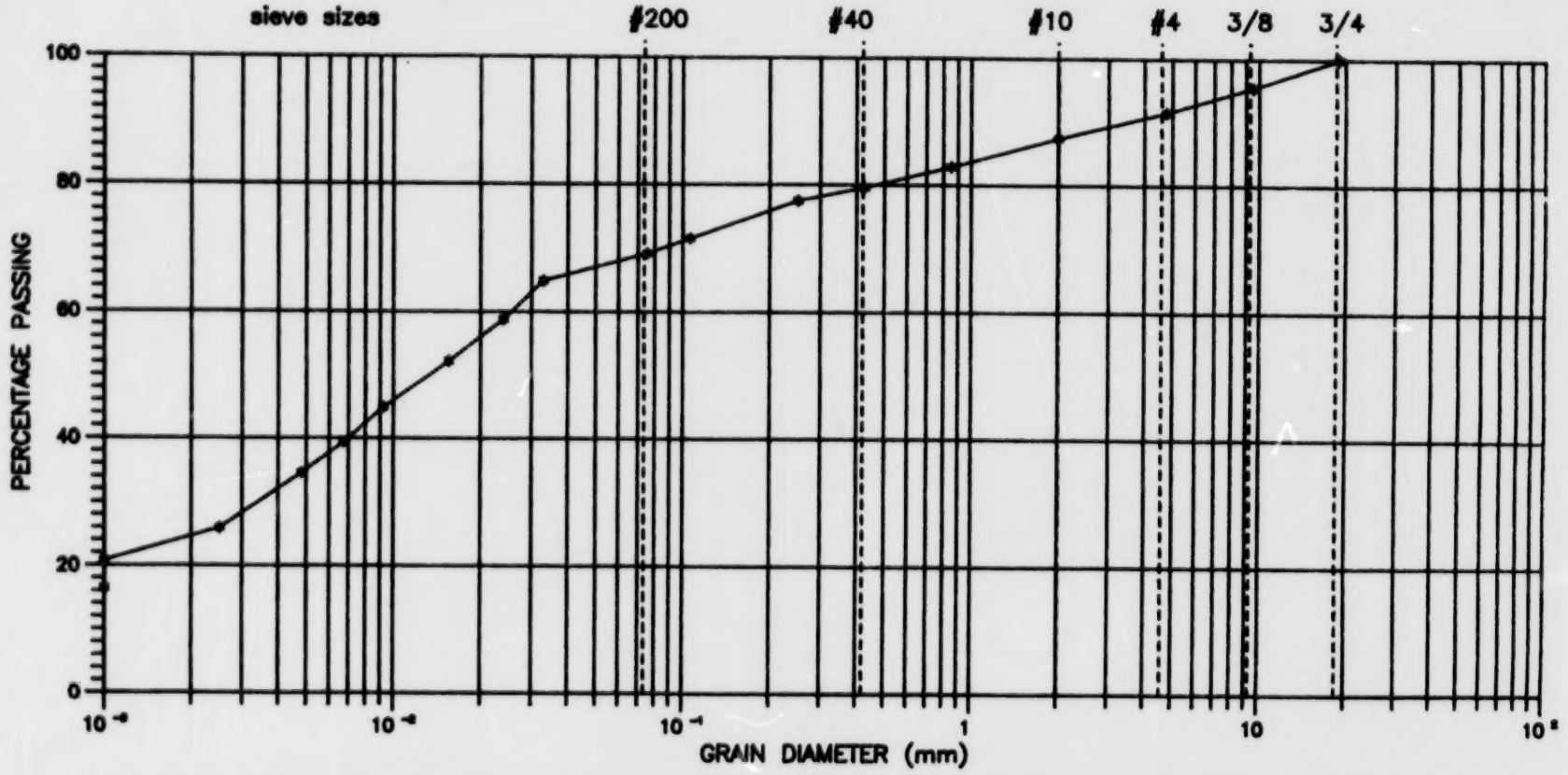
ONTARIO HYDRO
RESEARCH DIVISION

MECHANICAL ANALYSIS – MILTON T.S. BH–3 S–1



ONTARIO HYDRO
RESEARCH DIVISION

MECHANICAL ANALYSIS – MILTON T.S. BH–3 S–3



ONTARIO HYDRO
RESEARCH DIVISION

APPENDIX B

LOAD TEST ARRANGEMENT AND TEST RESULTS

ontario hydro
research division

FOR ONTARIO HYDRO USE ONLY
Unless release outside is authorized
by a director.

TO: Mr. T.J. Carmichael
Section Head - Soils Section
Civil Research Department

TESTS ON TRANSMISSION TOWER FOUNDATIONS

A test program to study the group effect of closely spaced augered transmission tower foundations was carried out at Ontario Hydro's Milton yard. Table 1 presents a description of the footings and tests conducted while Figure 1 shows the layout of the footings tested. The tests started on March 17 and were completed on March 20, 1987.

Test Arrangements and Test Procedures

The schematics for performing the uplift tests on paired and single footings are shown in Figures 2 and 3 respectively. Figures 4 to 7 depict the actual test arrangements. A pair of 2700 kN capacity, centre hole, hydraulic jacks were used to apply the uplift load to the footings. The jacks sat on bearing plates which were placed on top of two 48" WWF beams that rested on steel pads made from 20" WWF beams. The uplift force to the footings was transmitted by steel pulling rods connected to a pair of #18 Dywidag reinforcing steel bars embedded in each footing. A load cell on top of the hydraulic jack was used to monitor the amount of applied load to each footing.

All footing displacements were obtained by using linear voltage displacement transformers (LVDT's) attached to steel batter boards placed beneath the loading beams. Two LVDT's, one on either side of each footing, were used to obtain the average footing uplift. The ends of the batter boards rested on wood timbers placed well away from the zone of influence. Dial gauges attached to the same batter boards were used for back-up. A X-Y recorder plotted the load versus uplift performance for each footing. Figure 8 shows the instrumentation used to obtain the load versus displacement curves for all the tests.

job	file	date	report no.
24-257-40131	824.52	August 7, 1987	BG87-34-H

The test procedure for all the tests involved loading the single or each of the paired footings continuously at a rate of 200 kN per minute up to a load of 1000 kN. After reaching a load of 1000 kN, the loads were then increased in steps of 100 kN after which they were held for a few moments for all displacements to subside. The tests were terminated when the footings exhibited uplift failure or when the applied loads reached the safe working load capacity of the loading beams or the pulling rods.

a) Uplift Test on Paired Footings (Test No. 1)

Test No. 1 consisted of performing an uplift test on paired footings #5 and #6, shown in Figure 2. The clear span between the steel pads was 10 m. The uplift force to each footing was transmitted by a 3-1/2 inch diameter pulling rod attached to a yoke plate connected to a pair of Dywidag rebars embedded in the concrete footing (Figure 6).

At a load of 1700 kN, the load was maintained for 16 minutes after which the next load increment caused footing #6 to uplift in excess of 25 mm and the LVDT's and dial gauges had to be removed. Both footings were pulled up approximately 100 mm to observe the mode of soil failure after which the loads were released and the test was curtailed.

(b) Uplift Test on a Group of 4 Footings (Test No 2)

Four footings (#1,#2,#3,#4) were tested as a group simultaneously, such that footings #1 and #2 were loaded as a pair in tension while footings #3 and #4 were loaded as a pair in compression. The schematic for the tension test arrangement was identical to the one already described in Test No. 1, except that a pair of 36" W steel loading beams, 9 m long, were used instead of the 48" WWF steel beams. Thus, the clear span between the steel pads was 6.7 m.

The schematic of the compression test arrangement is shown in Figure 9. Figures 10 to 14 depict the actual test arrangement used for this group test. The compression loading was applied by two 8900 kN capacity hydraulic jacks set on bearing steel plates placed on top of footings #3 and #4. The jacks reacted against a pair of 48" WWF steel beams tied down at either end with Dywidag rebars to footings #7 and #8. Wood timbers were used to support the 48" steel beams before the loads were applied. A load cell placed between one hydraulic jack and the steel beams was used to monitor the applied compression load to the footings.

The displacement of each compression footing was recorded in the same manner as described in the uplift tests. Uplift of each reaction footing, #7 and #8, was also monitored by a pair of

LVDT's, (Figure 14). Two X-Y, a X-Y-Y as well as a cassette recorder were employed to plot the load versus displacement curves.

The load to each pair of footings was applied in the same manner as was described above. The test was terminated after reaching a load of 1800 kN per footing, the maximum safe working load capacity of the loading beams.

(c) Uplift Test on Paired Footings (Test No 3)

Footings #3 and #4, which had been tested in compression in Test No. 2, were then subjected to a tension test. Both footings were pulled simultaneously. The same test set-up and procedures were used to test these footings as were described above.

(d) Uplift Test on Single Footings (Tests No 4 and No 5)

Each of the reaction footings, #7 and #8, used in Test No. 2, was tested separately in uplift. Figures 3 and 7 depict the schematic and actual test set-up for these uplift tests respectively. Two 2700 kN capacity hydraulic jacks were used to apply the uplift load to each footing via two #18, Grade 60, Dywidag rebars. Both jacks sat back to back on a steel plate which was placed on top of the 48" WWF steel beams. The clear span between the steel pads was 8.5 m and 16.8 m in Test No. 4 and No. 5 respectively.

The test procedure for both footings was identical to the one already described. The test on each footing was terminated when the total load reached 90 percent of the minimum tensile strength of both #18, Grade 60, Dywidag pulling bars.

Test Results and Observations

The test results are summarized in Table 1. Figures 15 to 19 inclusive depict the load versus displacement curves for all the tests.

In Test No 1, both footings #5 and #6 exhibited pullout failure at the maximum load of 1800 kN. Figures 21 and 22 show the failure mode of both footings.

In Test No. 2 some difficulty was experienced trying to apply an equal load to both compression footings (#3 and #4) while at the same time applying an equal load on both tension footings when using the same hydraulic pump. The difficulty was resolved by using a separate pump for the compression footings and a separate pump for the tension footings. Since the two 8900 kN capacity jacks used for applying the compression load were not a matched pair, the loads were kept fairly constant by using a

load cell on one jack and monitoring the hydraulic pressure to the other. This operation required a separate control valve for each 8900 kN jack. In the future, each hydraulic jack should have its own load cell. While compression footing #3 exhibited approximately 8 mm of displacement at the maximum applied compression load of 1800 kN, almost no displacement was recorded on footing #4. The displacement was checked in several instances and was confirmed by a dial gauge. In consultation with the Civil Research Department, it was determined that footing #4 was installed directly above a boulder which prevented all displacement from the applied loads.

In Test No. 3 both footings #3 and #4 which were already subjected to a compression load in Test No. 2, uplifted in excess of 25 mm at the maximum applied tensile load of 1800 kN.

Uplift tests on the single reaction footings, #7 and #8, showed relatively small displacements for the maximum applied loads. In the future, if higher pullout loads are anticipated, a cluster of 4 or more #18, Grade 60, Dywidag rebars can be embedded in the footing and the test could be carried out by connecting all the Dywidag rebars to a steel plate and using a single pulling rod together with a 8900 kN capacity, hollow ram, hydraulic jack.

Approved by:

J. Kalnins
Unit Head - Work Equipment
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Mechanical Testing and
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Mechanical Research Department

Submitted by:

G.A. Senkiw
Associate Research
Engineer
Mechanical Testing and
Development Section

GAS:llg

TABLE 1

SUMMARY OF TESTS AND TEST RESULTS

Test No.	Date of Test	Test Description	Footing*	Depth (m)	Maximum Applied Load kN	Resulting Displacement mm	Figure No
1	Mar 17/87	Uplift test on paired footings #5 and #6	#5	5.59	1800	>25	15
			#6	5.59	1800	>25	
2	Mar 19/87	Group test of 4 footings: Paired footings #1 & #2 in tension	#1	5.49	1800	>30	16
			#2	5.59	1800	22	
		Paired footings #3 and #4 in compression	#3	5.18	1800	8	17
			#4	5.64	1800	0.2	
3	Mar 19/87	Uplift test on paired footings #3 and #4	#3	5.18	1800	29	18
			#4	5.64	1800	>25	
4	Mar 20/87	Uplift test on footing #7	#7	5.59	2800	20	19
5	Mar 20/87	Uplift test on footing #8	#8	5.59	2300	13	19

* all footings 0.97 m in diameter

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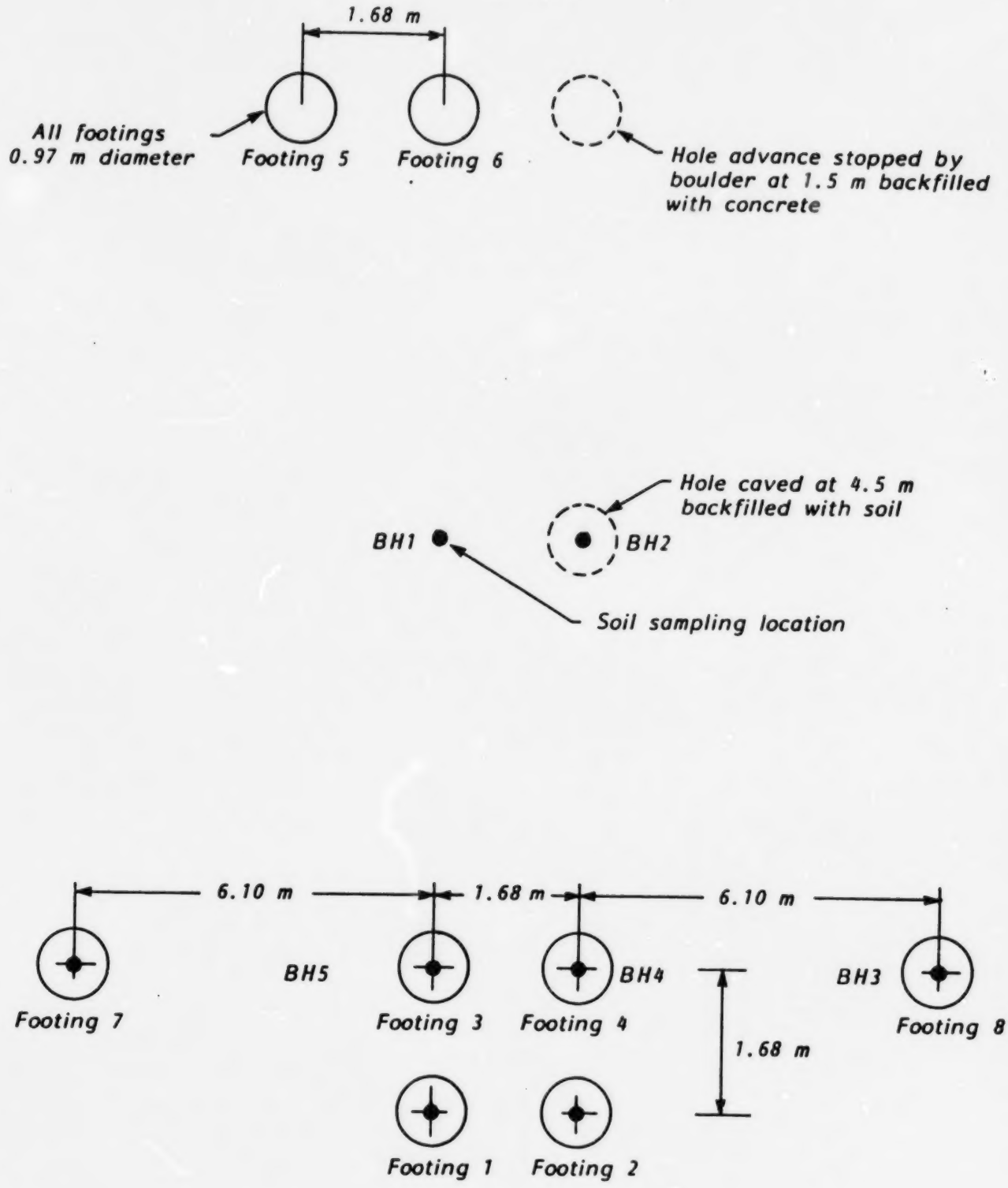

FIGURE 1
FOOTING LOCATIONS AT TEST SITE

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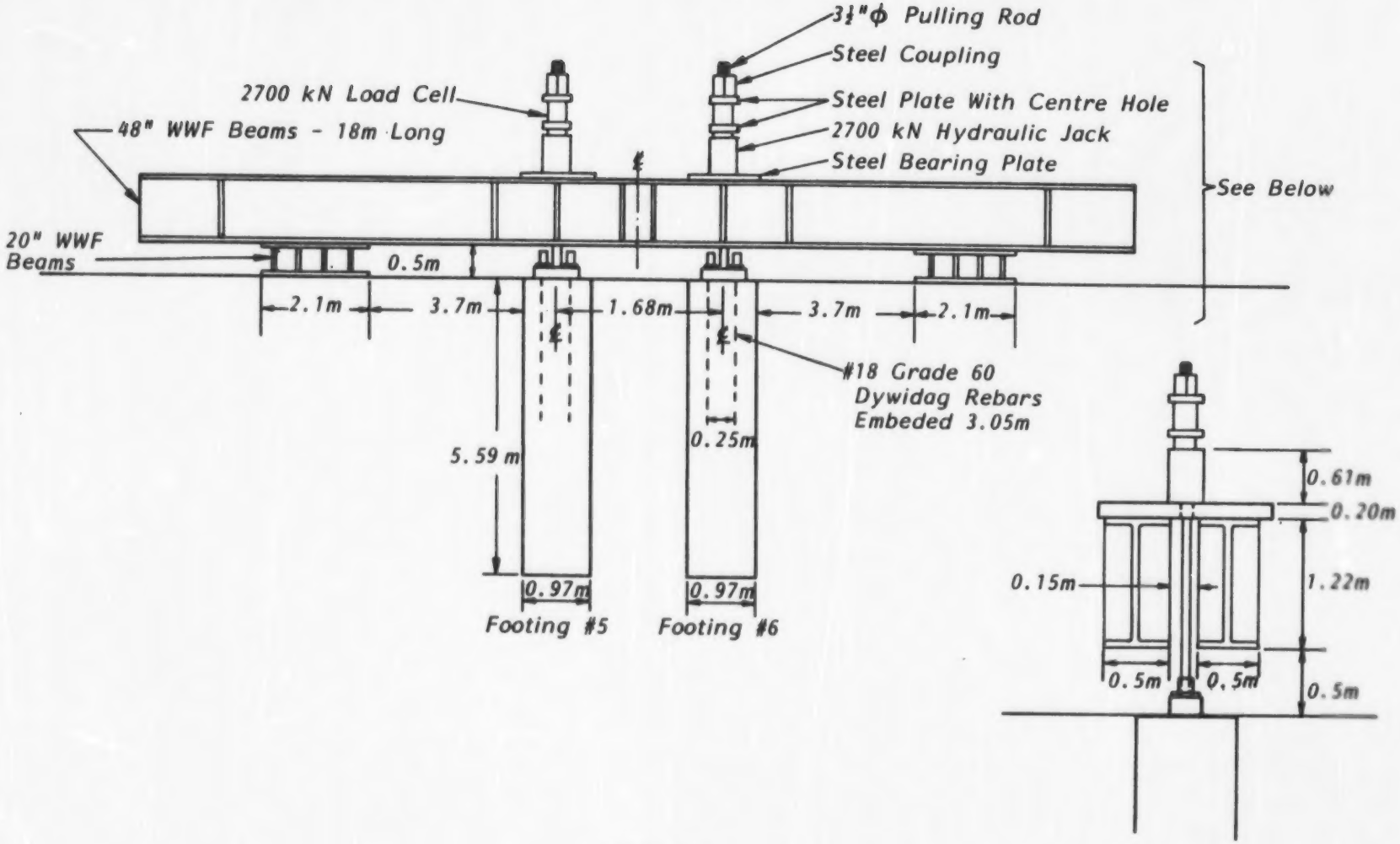

FIGURE 2
UPLIFT TEST ARRANGEMENT FOR PAIRED FOOTINGS,
#5 & #6 (TEST NO 1)

175186-RD
BG87-134

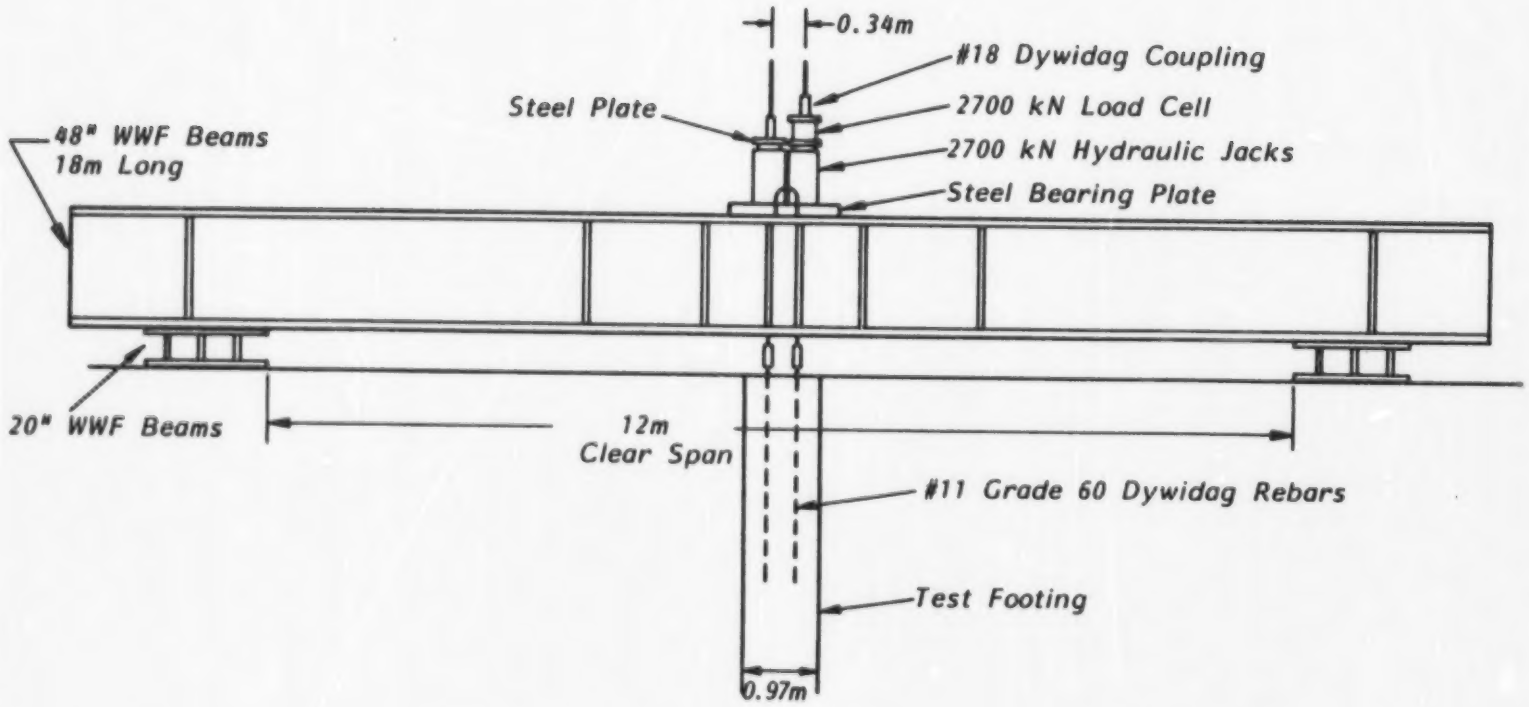

FIGURE 3
UPLIFT TEST ARRANGEMENT FOR SINGLE FOOTINGS,
#7 & #8 (TEST NO'S 4 & 5)

175193-RD
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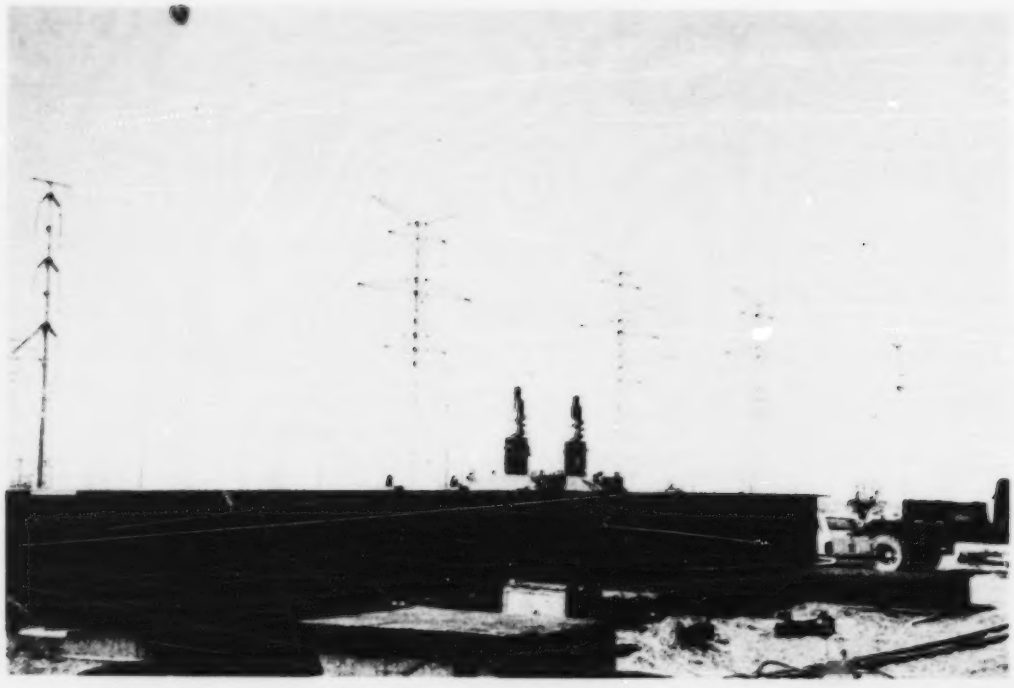

FIGURE 4

UPLIFT TEST ARRANGEMENT
FOR PAIRED FOOTINGS #5 & #6, TEST NO 1

FIGURE 5

PAIRED FOOTINGS UNDER TEST LOAD
IN TEST NO 1

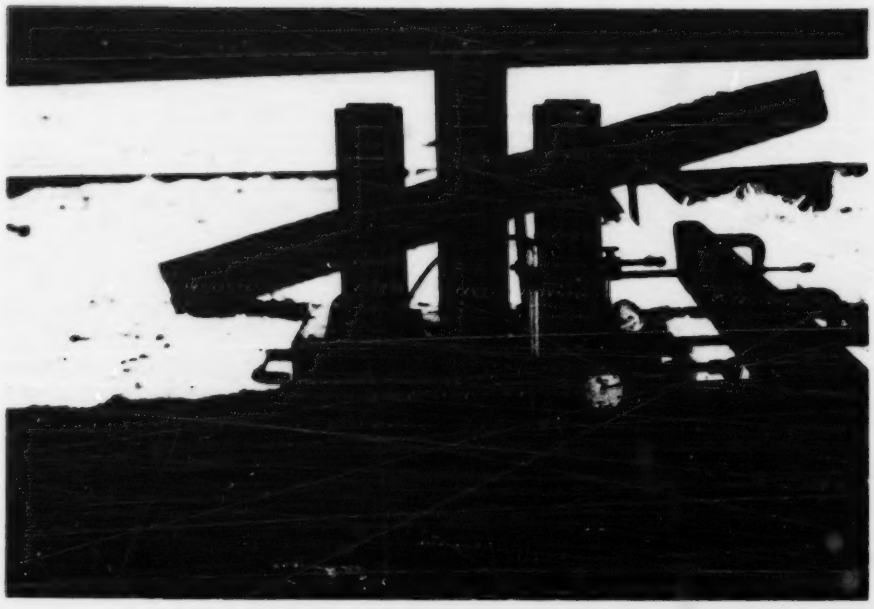

FIGURE 6
CLOSE-UP OF FOOTING CONNECTION AND
UPLIFT INSTRUMENTATION FOR TEST NO 1

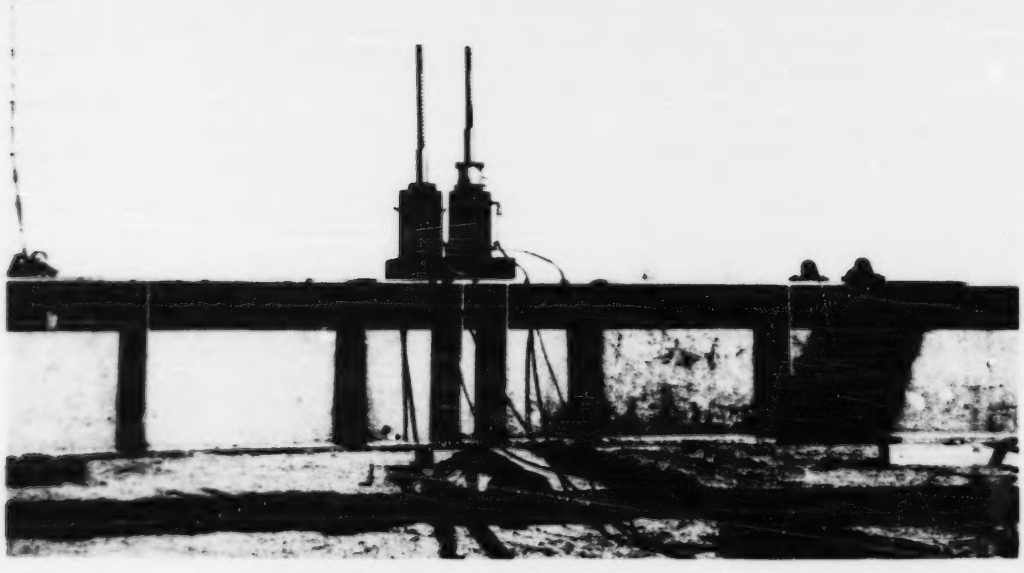

FIGURE 7
UPLIFT TEST ARRANGEMENT ON
SINGLE FOOTING TEST NO 4

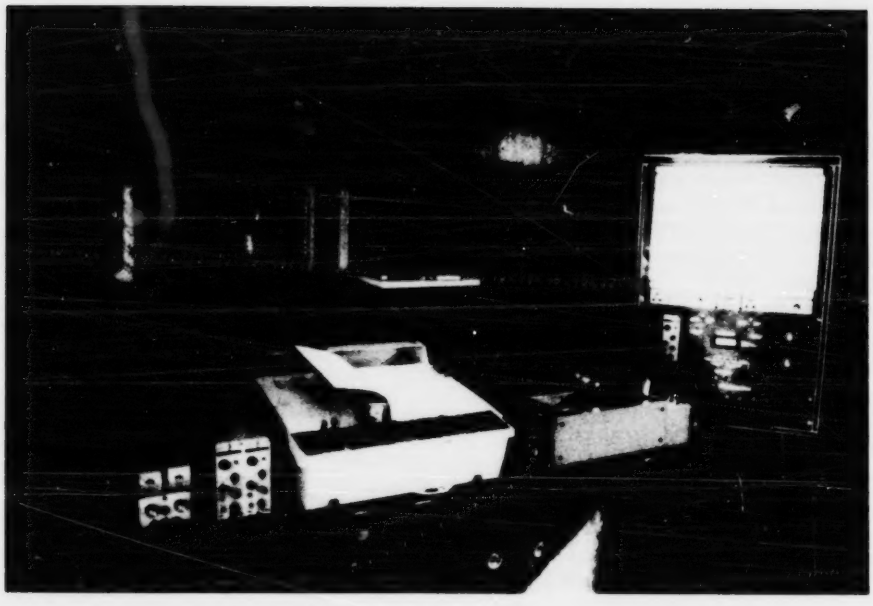

FIGURE 8

TEST APPARATUS FOR OBTAINING
LOAD - VERSUS DISPLACEMENT CURVES
ON ALL THE FOOTING TESTS

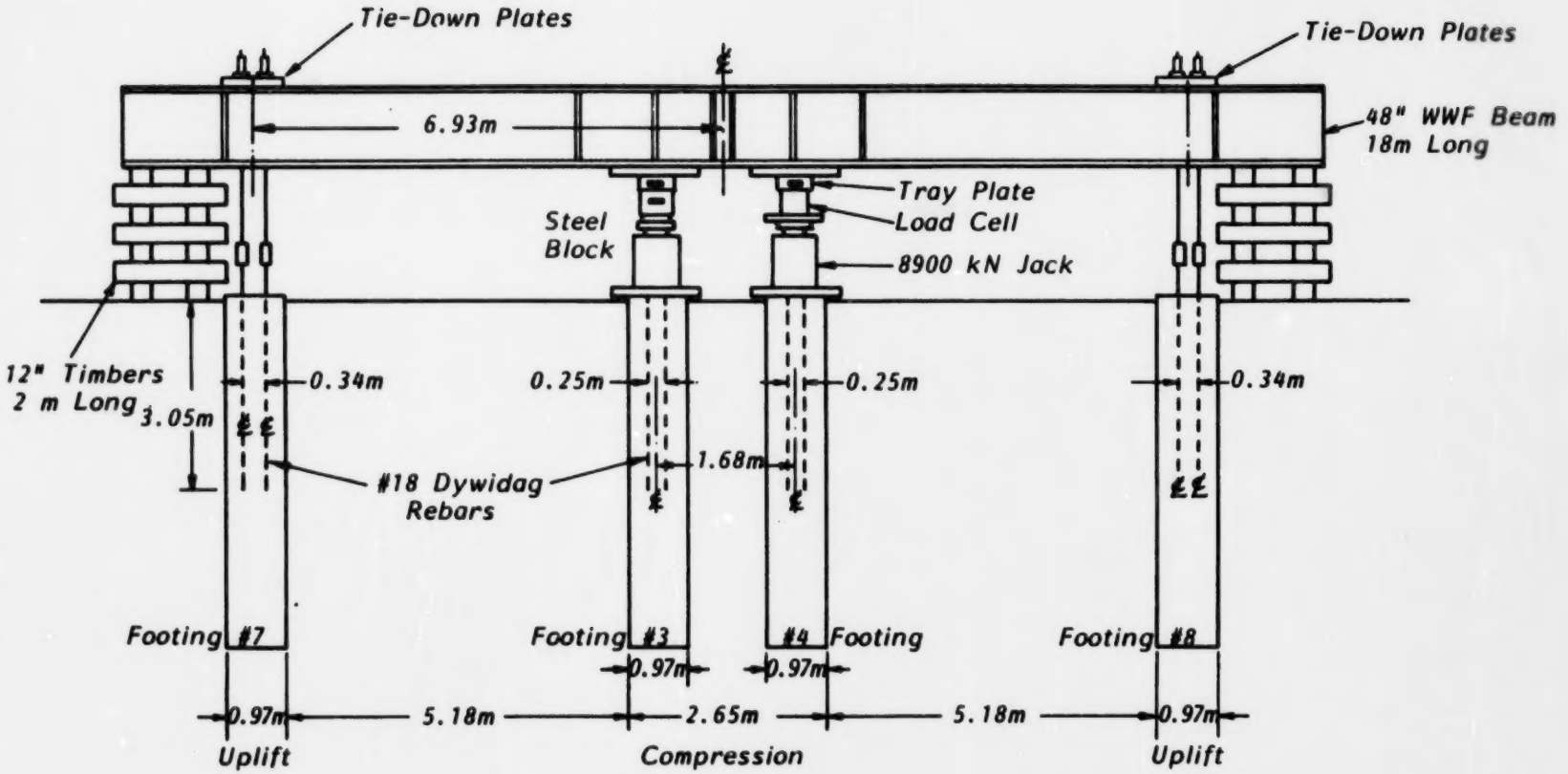

FIGURE 9

COMPRESSION TEST ARRANGEMENT IN GROUP OF 4 FOOTINGS, TEST NO 2

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87-134

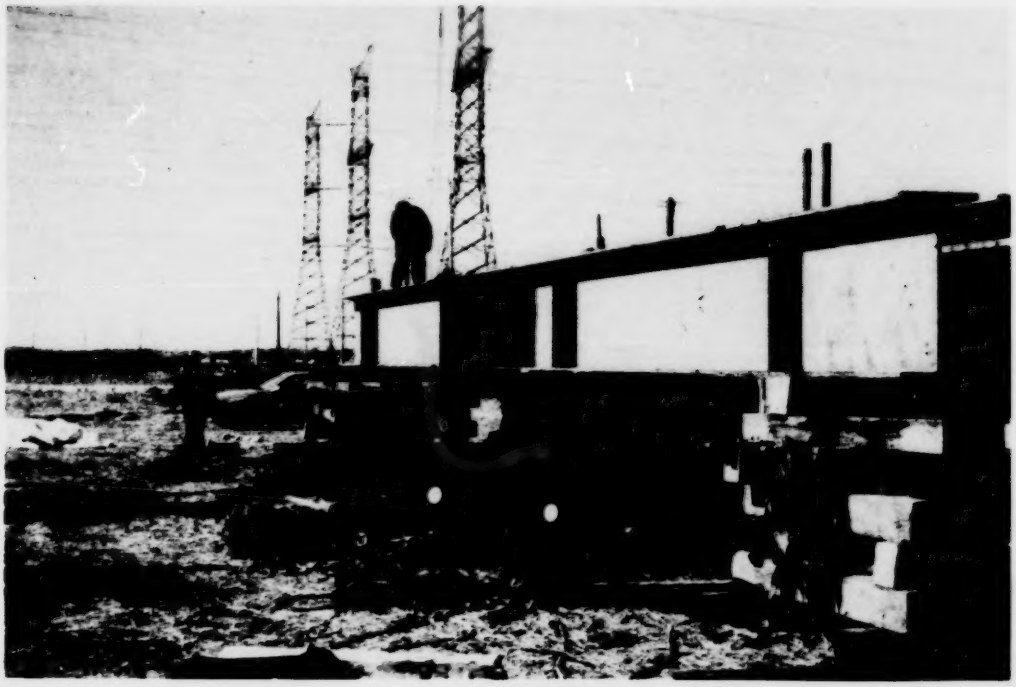

FIGURE 10

TEST ARRANGEMENT FOR TESTING A GROUP OF
4 FOOTINGS - 2 IN TENSION AND 2 IN COMPRESSION
SIMULTANEOUSLY, TEST NO 2

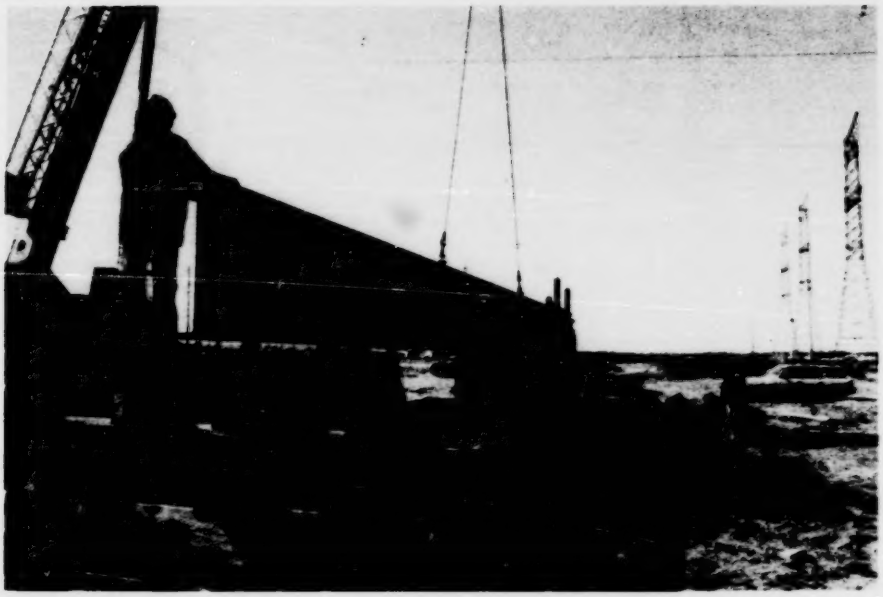

FIGURE 11

TENSION TEST SET-UP FOR TEST NO 2

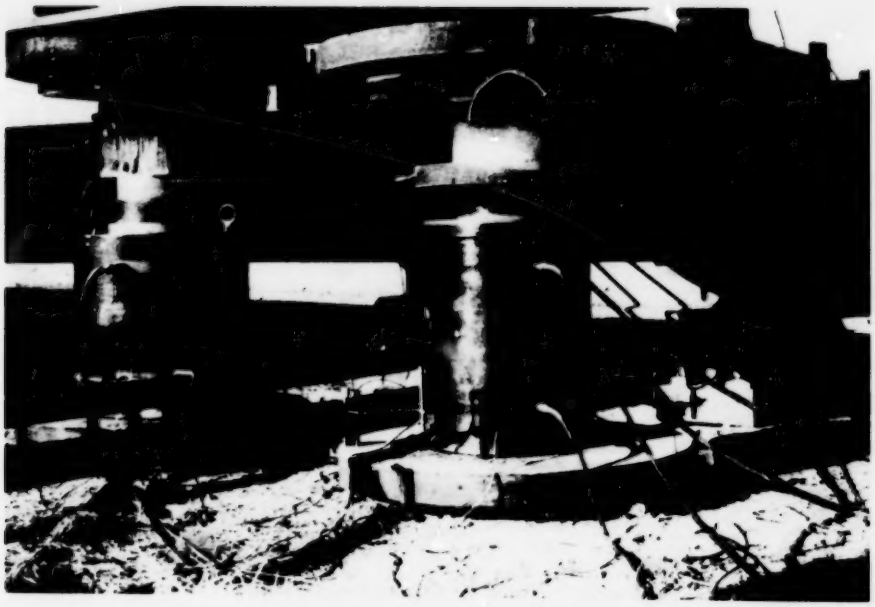

FIGURE 12
COMPRESSION TEST SET-UP FOR TEST NO 2

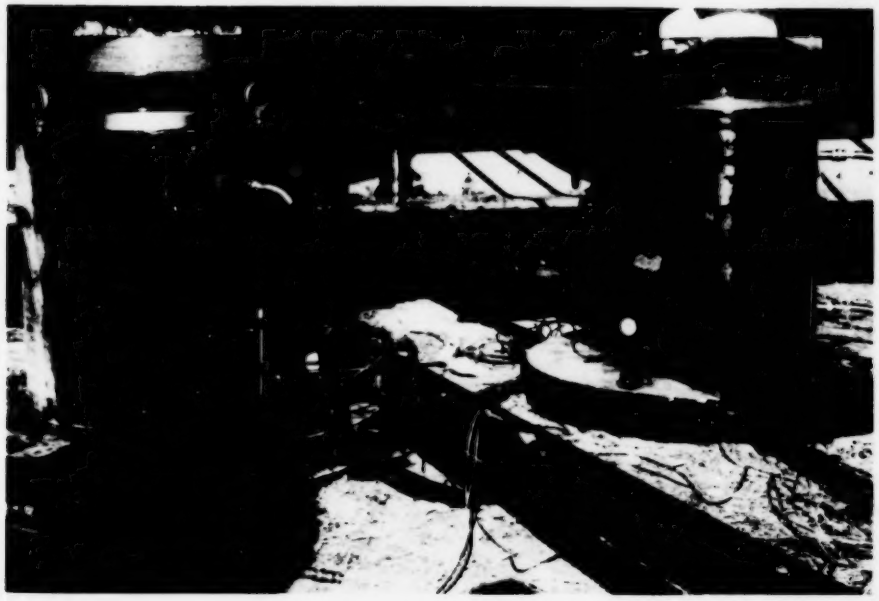

FIGURE 13
INSTRUMENTATION FOR MEASURING
COMPRESSION DISPLACEMENT IN TEST NO 2

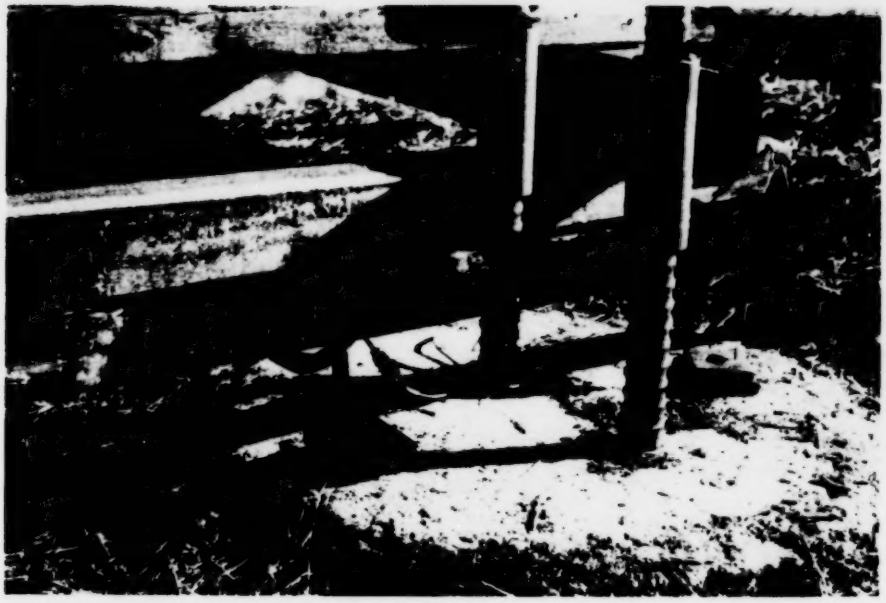

FIGURE 14

MONITORING UPLIFT ON REACTION FOOTING
DURING TEST NO 2

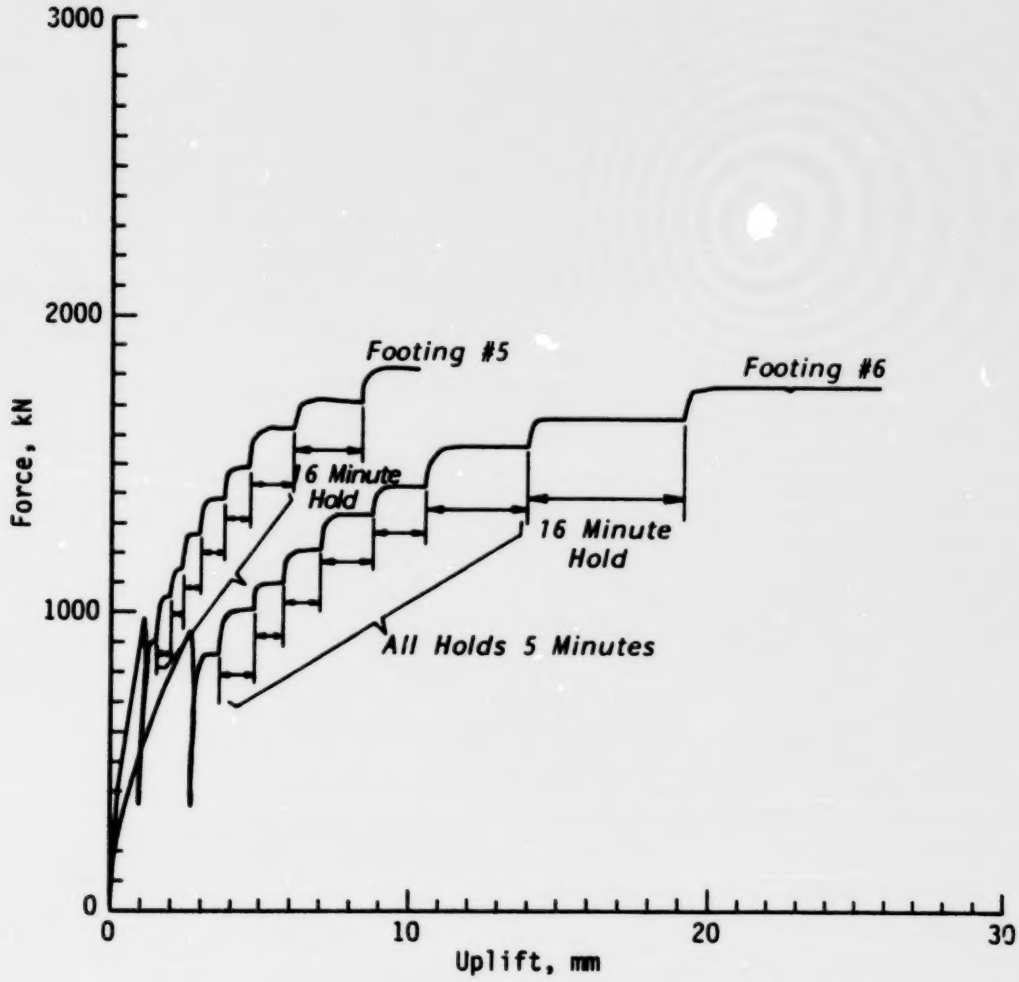

FIGURE 15

UPLIFT OF FOOTINGS #5 & #6 (PULLED AS A PAIR) TEST NO 1

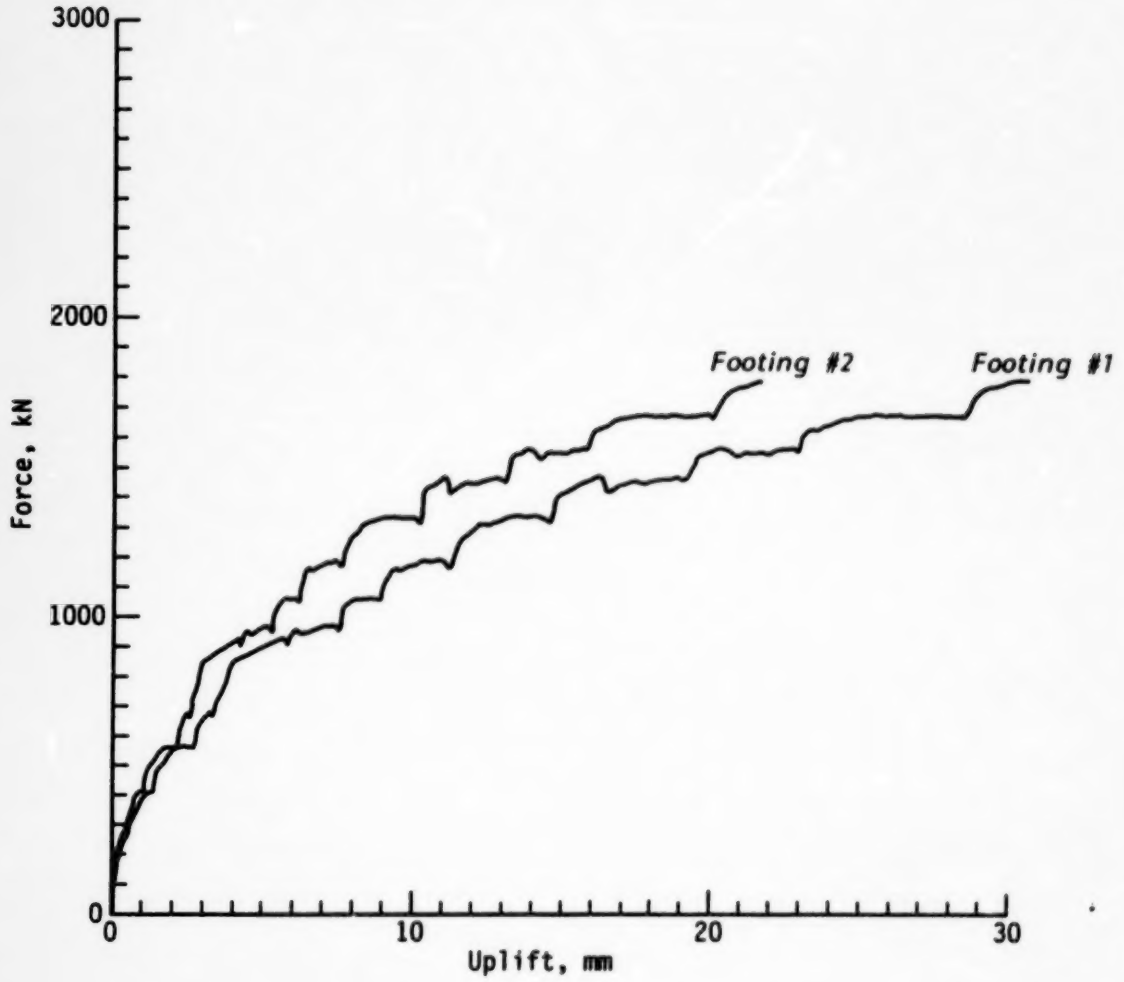

FIGURE 16

UPLIFT OF FOOTINGS #1 & #2, GROUP OF FOUR FOOTINGS, TEST NO 2

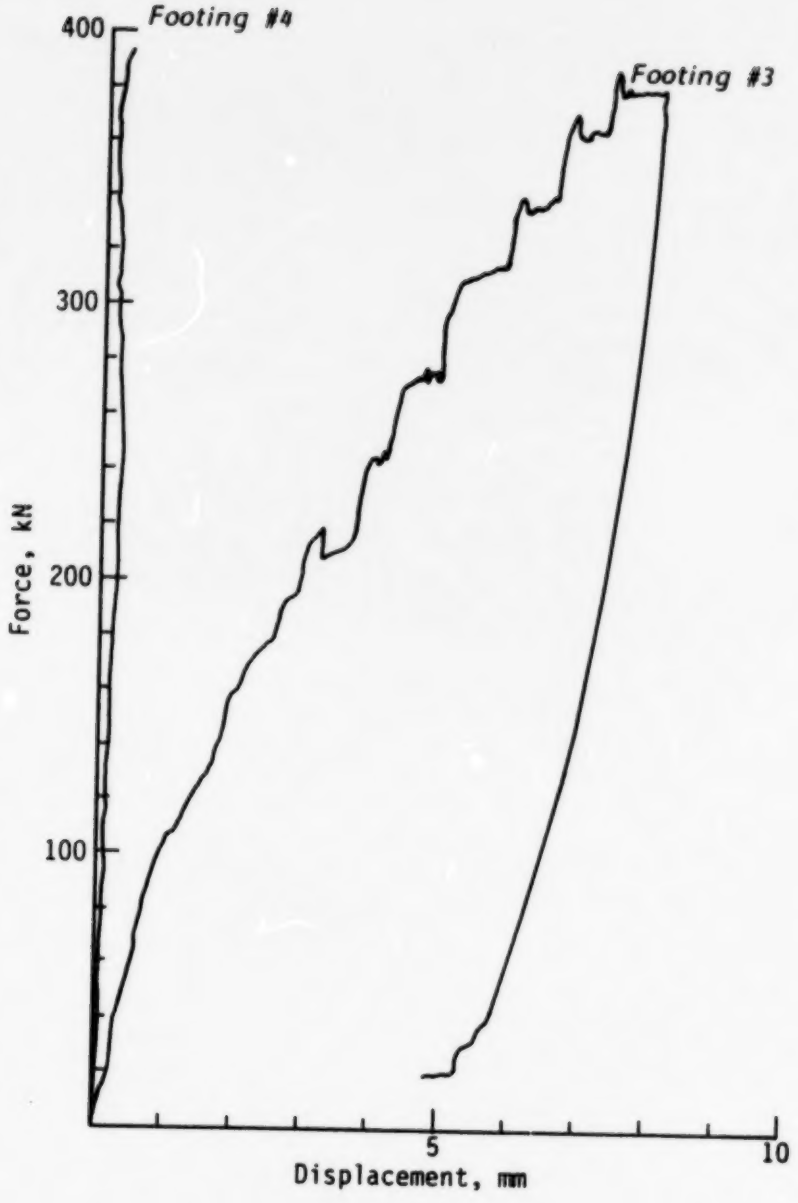

FIGURE 17
DISPLACEMENT OF FOOTINGS #3 & #4, GROUP OF FOUR FOOTINGS, TEST NO 2

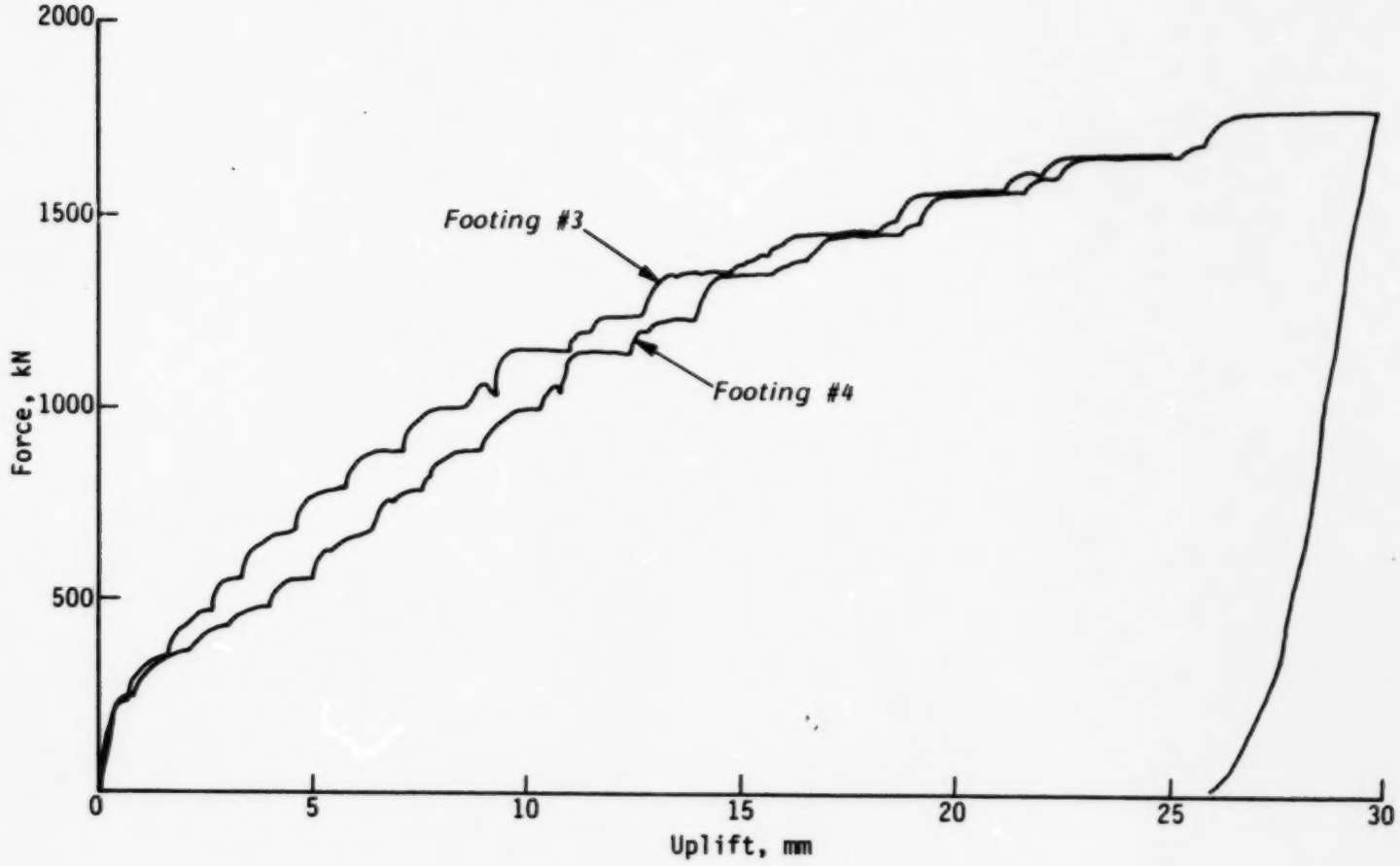

FIGURE 18
UPLIFT OF FOOTINGS #3 & #4 (PULLED AS A PAIR) TEST NO 3

175197-RD
BG87-34
87-134

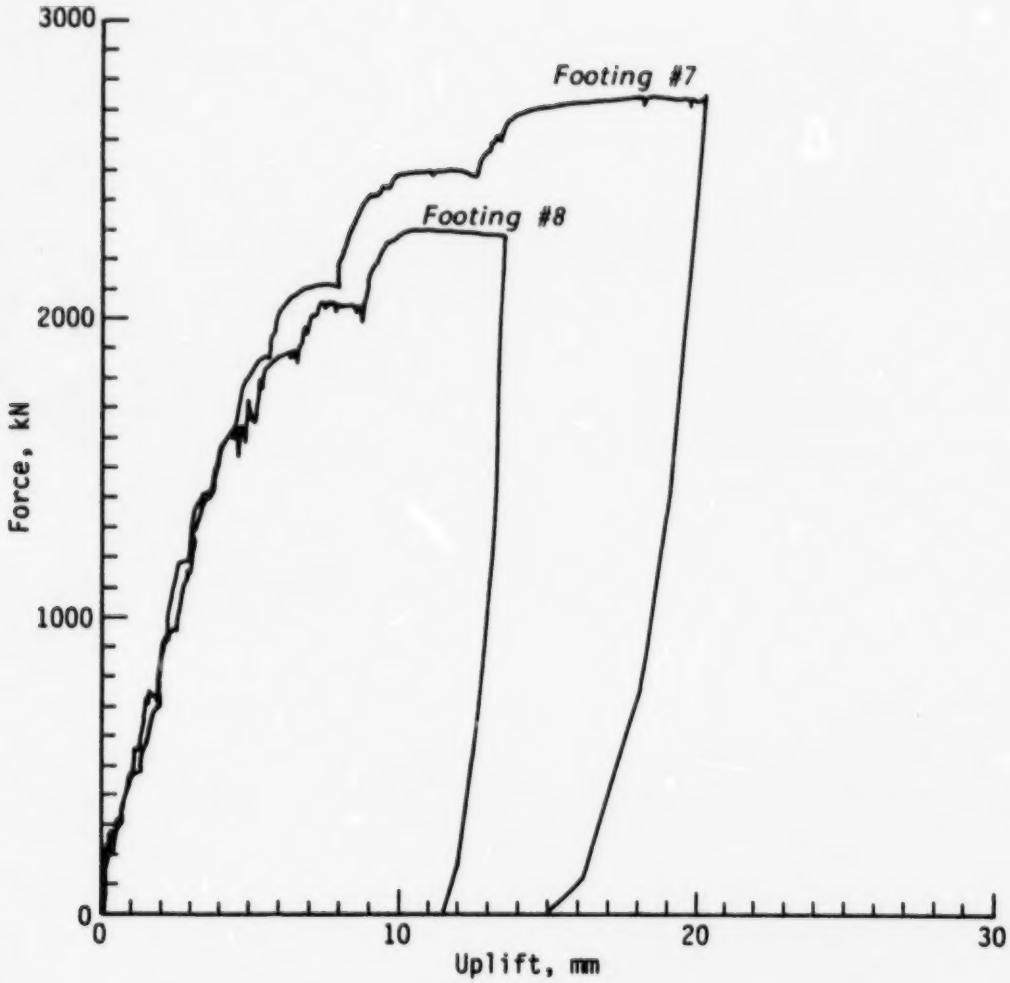

FIGURE 19
UPLIFT OF FOOTINGS #7 & #8 (EACH WAS TESTED SEPARATELY) TEST NO'S 4 & 5

FIGURE 20

UPLIFT ON PAIRED FOOTINGS
TEST NO 1

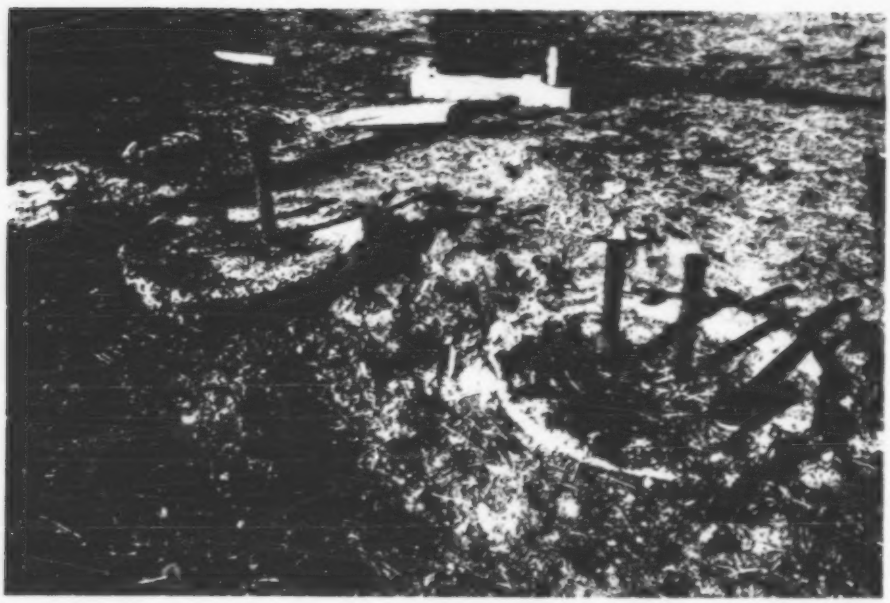

FIGURE 21

PAIRED FOOTINGS AFTER TEST NO 1

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